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The 328-Ft. Reinforced-Concrete Arch Bridge Across the Tiber at Rome.

The new Ponte del Risorgimento, a single reinforced-concrete arch of 328-ft. span, across the Tiber at Rome, Italy, which was opened for traffic in May of this year, is the longest masonry arch span so far attempted, surpassing by 33 ft. the hitherto longest stone voussoir arch, that of Plauen, in Germany, and by 8 ft. the record concrete arch, in the Grafton bridge at Auckland, N. Z. Notwithstanding its size it is of the lightest type of construction and of a design peculiar to certain European engineers, but rarely attempted on this side the Atlantic. The slenderness of the members, the insecurity of the footings and the special falsework all add to the interest of the structure.

The bridge was first projected in May, 1909, as a crossing from the city of Rome to an exposition on the other side of the Tiber, now in progress. On account of lack of funds and the very short time available it was at first decided to build the structure of wood, but the local authorities, who had for some time contemplated a permanent crossing at this location, offered to cooperate with the exposition officials, with the result that a reinforced-concrete design was adopted. Many studies of the project were made and the design on the Hennebique system, submitted by "La Societta Porcheddu Ing." was accepted by the special commission appointed to supervise the work. The contract under this design was awarded to a firm composed of Messrs. Porcheddu and Bentivegna for the sum of 1,250,000 lire ($238,000) and work was started on Nov. 11, 1909, under the personal direction of Mr. G. A. Porcheddu.

FIG. 1. THE PONTE DEL RISORGIMENTO; THE LONGEST MASONRY ARCH IN THE WORLD.
(Built of Reinforced Concrete Across the Tiber at Rome, Italy. Span, 328.1 ft., Rise, 32.8 ft.)

Design.

The clear opening of the arch is 100 m. (328.1 ft.) and the rise 10 m. (32.8 ft.), and the crown of the intrados is 47 ft. above mean water of the river. As shown in Fig. 2, the structure is really a cellular box, with a solid lower face, which is the arch proper, a solid top plane, which is the floor slab, and seven longitudinal vertical walls braced together by vertical transverse walls at intervals and by transverse beams. This whole structure rests at either end on a concrete slab founded on concrete piles and abuts against a cellular abutment similar in design to the arch proper, and founded also on concrete piles. It carries a roadway 13 m. (42.6 ft.) wide, flanked on either side by a 3.5 m. (11.5 ft.) sidewalk, all three of which are widened somewhat at the approaches.

The arch rib is 20 m. (65.6 ft.) wide, and is composed of a central portion of varying width whose transverse elements are parallel to the plane of the floor, with a beveled face on each side extending from the main portion up to the outside spandrel walls. This detail may be noted on Fig. 2 and its appearance is shown in the view in Fig. 1. The rib is very shallow, being but 20 cm. (8 ins.) thick at the inner panels and 28 cm. (11¼ ins.) at the other panels of the crown and 50 cm. (20 ins.) at the springing line. It is reinforced with longitudinal, transverse and vertical rods, as shown in Fig. 3. At the spring it abuts against a transverse beam, which is in reality an enlargement of the arch rib proper, and this beam in turn abuts against the longitudinal walls of the cellular abutment, the thrust being thus transmitted to the base of the other abutment.

In regard to provisions for expansion in the bridge, our information is varied. The latest issue of the "Annales des Ponts et Chaussees" states that one of the remarkable features of the structure is the absence of "any special device, such as hinges, expansion joints, etc., to take care of the changes in form due to temperature." This statement is confirmed by the available drawings, which show the arch and walls unbroken from end to end. On the contrary, the Italian technical journal, "L'Ingegneria Ferroviaria," states that in each longitudinal vertical wall there are six expansion joints, in which the concrete is removed for about an inch, the space being filled with "cork," and across which the steel reinforcement is continuous. However, these joints do not appear on the drawings which accompany the article.

The main longitudinal walls in the arch proper are 20 cm. (8 ins.) thick and are tied in, by the reinforcement, to both the arch ribs and the floor slabs. From the crown to a point about one-third way down the arch the floor slab is carried on these main longitudinal walls and on intermediate longitudinal walls of lesser thickness, but from that point down to the springing line transverse floor beams are provided to reduce the floor slab spans between the spandrel walls. The location and reinforcement of the diaphragm walls and transverse beams are best shown on Fig. 3. On Fig. 2 will also be noted the rear wall of the cellular structure of the arch proper, which forms a retaining wall against the earth fill in the cellular abutment.

The design of the abutments is shown in Figs. 2 and 4. Each consists of a forward portion, comprising a slab resting on concrete piles and a rear portion consisting of seven longitudinal walls, continuations of the spandrel walls of the arch, each footing at intervals on concrete piles and all interbraced with beams and walls. All of the rear portion is filled with earth up to floor level. The longitudinal walls are 30 cm. (12 ins.) thick, as compared with the 20-cm. (8 in.) arch walls which they continue. On account of the extremely soft nature of the ground on which the abutment is placed, it is protected against inflow from the river by a face wall of concrete sheet piles driven clear across the width of the abutment.

Construction.

ABUTMENTS.—The subsurface along the Tiber is of the weakest character and this was the cause of much doubt as to the possibility of an arch bridge. The problem was finally solved by the use of the Hennebique "compressol" system of piling and the cellular abutment described. The "compressol" system* of concrete piling consists in the use of a heavy conical plunger which is raised some height in the air and dropped successively into the ground, so that after a time it forms a circular pit with highly compressed walls. In sandy or water-soaked ground, artificial walls are formed by

*Engineering News, Sept. 27, 1900, p. 209.

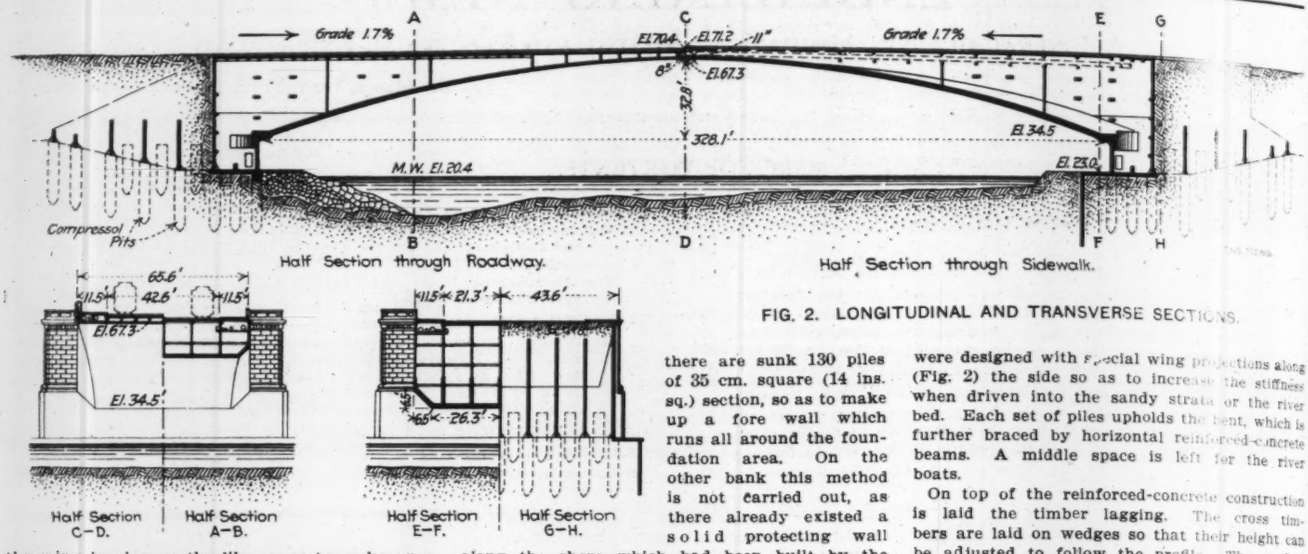


FIG. 2. LONGITUDINAL AND TRANSVERSE SECTIONS.

throwing in clay or the like so as to make up a stiff and water-tight tube. The base is made by using loose rock which is pounded down and spread out into the soil by the plunger so as to give a solid base. The pit is then filled in with concrete which is well pounded down. At the upper part, a reinforcing of vertical rods is used so as to join this part to the bridge structure. In this way the 72 pits for each abutment were laid, and the general base extends out considerably beyond the abutment area. There were also sunk 24 secondary "compressol" pits which were not reinforced with rods, in order to further consolidate the ground. Somewhat of a novelty is the use of a fore wall in order to protect the foundation against the action of the water. Owing to the loose nature of the ground, this could be carried out readily by the hydraulic process, using concrete piles having a large tubular opening throughout their length, so that after the piles are started in the ground they are then sunk by water jet. On the right bank there are sunk 130 piles of 35 cm. square (14 ins. sq.) section, so as to make up a fore wall which runs all around the foundation area. On the other bank this method is not carried out, as there already existed a solid protecting wall along the shore which had been built by the city engineers many years ago, and this gives sufficient protection on this side. The construction work for the foundations was somewhat modified here to provide for these conditions.

FALSEWORK.—Owing to the fact that the Tiber has a variable flow and is subject to very heavy freshets, a leading problem was to build a suitable centering for the bridge. The freshets occur in the period lasting from early in the Autumn to well on in the Spring, and every year there is a rise of 20 and even 25 ft. A falsework had therefore to be designed to provide for the force and duration of the flood. In order to ensure the stability of the falsework during these floods, reinforced concrete was used for the main part of the centering. A view of this falsework is given in Fig. 3. It consists of eight transverse bents, each comprising seven columns located each under one of the seven longitudinal walls in the arch, and each footing on a frame of 16 piles driven into the river bottom. The piles were designed with special wing projections along (Fig. 2) the side so as to increase the stiffness when driven into the sandy strata of the river bed. Each set of piles upholds the bent, which is further braced by horizontal reinforced-concrete beams. A middle space is left for the river boats.

On top of the reinforced-concrete construction is laid the timber lagging. The cross timbers are laid on wedges so that their height can be adjusted to follow the profile. The method used by Mr. Porcheddu met with some skepticism on the part of engineers and constructors when it was first proposed, but after the scaffolding had been constructed it fully proved all that was claimed for it. One advantage is that special carpenter work is not needed, as once the piles are driven the rest can be done by the regular concrete workmen.

The principal advantage lies in the strong and continuous mass which the reinforced-concrete structure offers, and this was well brought out in the present case. On Dec. 29, 1910, a heavy river steamer was washed downstream during a freshet and struck the fore part of the middle upright. The shock was so violent that some of the piles were smashed, while the boat sunk to the bottom at once and the crew were barely saved. At this time the concrete work on the main arch of the bridge was only half completed, and this gave a very heavy load on the centering. After the underpinning broke, there was more than a

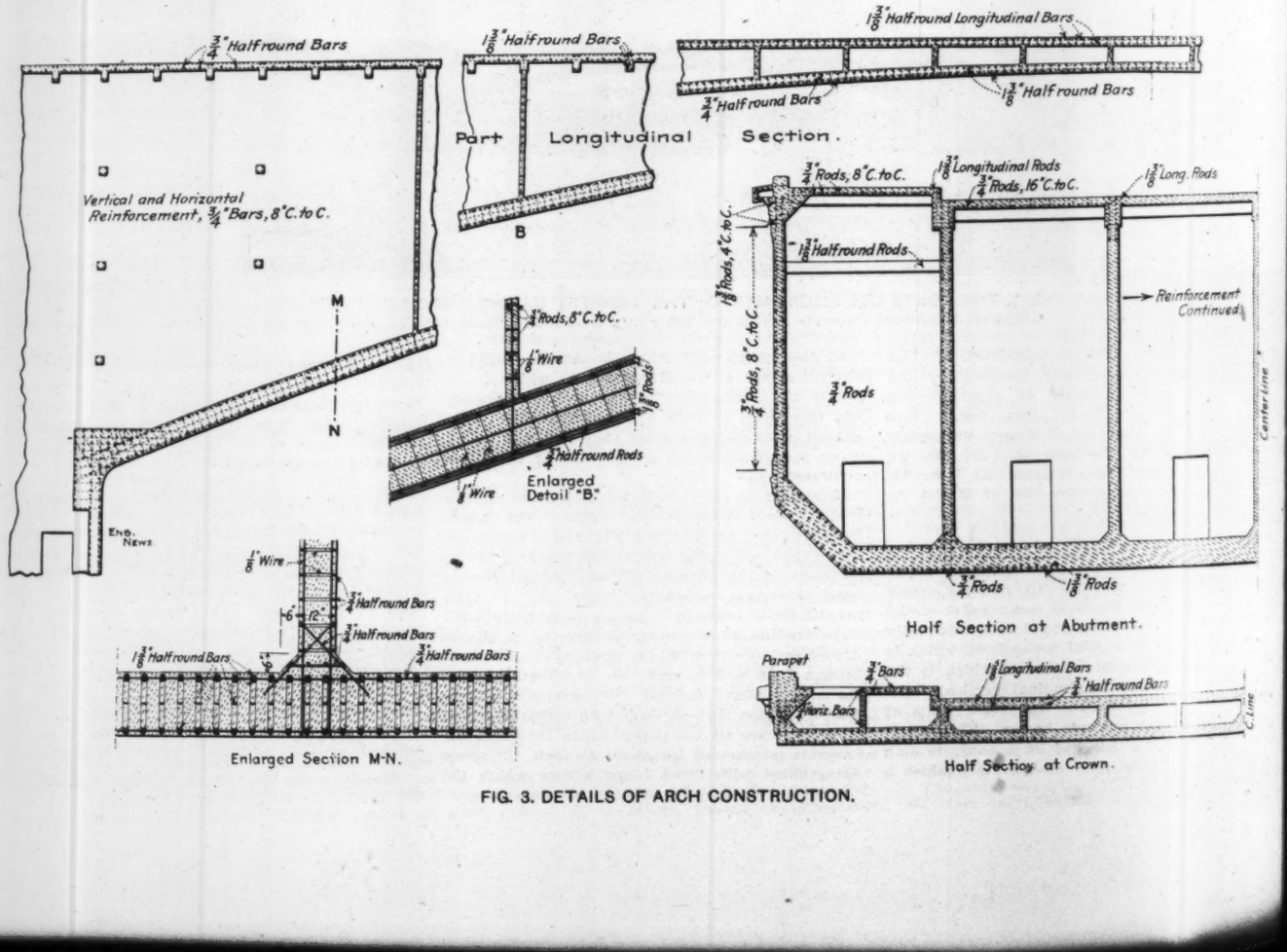


FIG. 3. DETAILS OF ARCH CONSTRUCTION.

10-ft. overhang at this part, but the falsework still continued to carry the load until the damage could be repaired. No harm was done to the concrete work of the arch. With an ordinary timber falsework such an accident might have been disastrous to the bridge.

For the reinforcement of the members of the bridge and falsework only two sizes of rods were used, one a ¾-in. rod and the other a 1⅜-in. rod. Both were after a special type designed by Mr. Porcheddu with a crescent section having lugs on both outer and inner circles reinforced with a double network of vertical and horizontal irons

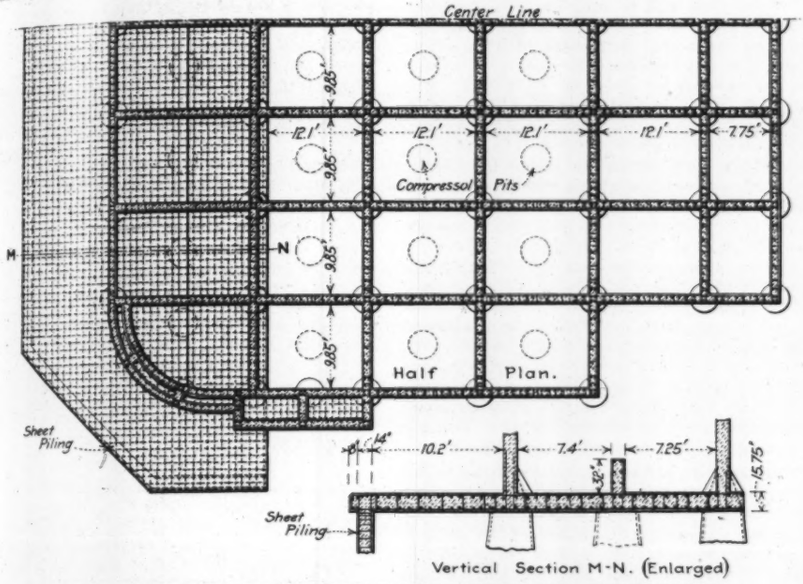


FIG. 4. HALF PLAN OF THE ABUTMENT; PONTE DEL RISORGIMENTO.

of equal section with frequent bindings throughout all the mass. All members were cast in place with the exception of the floor beams. These beams, 18 × 25 cm. (7.2 × 10 ins.), were made outside and put in during the construction, when the longitudinal beams were finished to the proper height.

The bridge is paved with slabs of compressed asphalt and cement. The decoration of the bridge is carried out in artificial stone which imitates travertine.

FIG. 5. VIEW OF BRIDGE DURING CONSTRUCTION, SHOWING CONCRETE FALSEWORK IN PLACE.

Loads.

The bridge was designed for a uniform live load of 500 kg. per sq. m. (100 lbs. per sq. ft.) plus three 16-ton road rollers running abreast.

After completion the bridge was tested as follows: A uniform load of 100 lbs. per sq. ft. was put all over each half of the bridge and then over the whole span. Three road rollers of 16 tons each were passed over abreast, and the same time the sidewalk was loaded at the above rate. In these tests the deflection at the center was not to be over 1/1,500 of the span, or 67 mm. (2.7 ins.) in the present case. In the static tests the bridge was loaded with sand and gravel and this somewhat exceeded the official figure, reaching 620 kilogrs. per sq. m. (124 lbs. per sq. ft.). The test showed only 32 mm. (1.3 ins.) deflection, or less than half the prescribed figure. Only one-half of the bridge was then loaded, and the deflection was 13 mm. (0.4 in.) in this case. In the dynamic tests, made with the road rollers, the indicating instruments showed that the deflection was not over 2 mm. (0.08 in.) and the readings all returned to zero, showing that the elasticity is perfect. The results of the tests were most satisfactory.

The Use of CO_2 to Regulate the Temperature of combustion in a gas-producer fuel bed which was patented by Mr. Henry L. Doherty (U. S. Patents Nos. 829,105 and 844,504) has been upheld in the Circuit Court of the United States, District of Massachusetts, by Judge Brown, in an equity suit brought by the Combustion Utilities Corporation against the Worcester Gas Light Co. for infringement.

The patents held by Mr. Doherty cover the process and the apparatus for introducing a mixture of air and CO_2 under the grates for the purpose of regulating the temperature of the fuel bed and holding it below that point at which clinkering takes place. The decision admits the prior art of introducing products of combustion with air under grates, but holds that this was for the purpose of utilizing the heat in the products of combustion or for diluting the oxygen with a large amount of inert gases which cooled the fuel bed. The decree finds that this method of reducing the temperature of the fuel bed was not sufficient to prevent the formation of slag and clinker up to the time of Doherty's invention. Mr. Doherty's claim of utilizing the CO_2, which absorbs heat in splitting up to CO and O, to balance the heat of combustion of the solid fuel to CO, is supported. Various patents granted to Siemens, Eustis, Biedermann & Harvey, and Eldred were reviewed, but not found to constitute the advance shown in Mr. Doherty's patents. The substance of the decision seems to be summed up in the following passage:

> A very protracted and careful examination of the record convinces me that Doherty made a meritorious advance in the art; that he was the inventor both of an apparatus and of a new process. While there is much ground for discussion in that part of his specification which relates to the theory of operation, and in his claims, yet it seems reasonably free from doubt that Doherty's apparatus for regulating combustion was a new structure which is not anticipated in the prior art, and that it embodies the discovery that by a definite control of the mixture of air and waste products of combustion and by a definite regulation of the temperature of his mixture, he could accomplish a valuable result which could not be accomplished, and had not been accomplished, merely by using the waste products of combustion and air without regard to their exact proportions, the thoroughness of mixture and the temperature at which the mixture should be introduced. Exact adjustment of these as means to the specific and novel end of maintaining temperature within a maximum limit, for the elimination of the undesirable slag which accompanies the use of the same means not exactly adjusted, and applied without regard to and without attaining this specific improvement, distinguishes Doherty broadly from the prior art.
>
> While the general resemblance of the means of generating producer gas brings the prior art close to Doherty, yet in the specific feature of successfully eliminating the formation of slag while operating a gas producer, which is the subject of his invention, there is, so far as the patents in evidence are concerned, no prior disclosure and no prior art with means which resemble Doherty's.

The Effect of Pigments on Linseed Oil.—In Engineering News, Oct. 5, 1911, a description of tests by Mr. H. A. Gardner, on the effect of pigments on linseed oil, was abstracted from the September issue of the "Journal of Industrial and Engineering Chemistry" of the American Chemical Society. The following letter from Prof. A. H. Sabin relative to this article appeared in the October issue of the same journal and is here reprinted as of interest in connection with our abstract:

> Sir: In Mr. Gardner's paper he says that in my tests barytes, etc., appeared to accelerate the drying of linseed oil films, and that "the point at issue seems to be whether the inert pigments really have any chemical action when enveloped in linseed oil."
>
> Reference to my brief paper shows that nothing was said about either of these matters, except to call attention to the curves and suggest that more might be known by future study, also to the fact that the effect of pigments "may perhaps" be due to hindering peroxidation, both of which are obvious. The paper is a record, and until the facts are questioned, there is no "point at issue."
>
> Very likely the effects which all pigments have on oil, whether as stimulators or inhibitors of oxidation or any other process, are due to catalytic or contact action. I think all pigments have some effect on oil, not depending on their fineness but on their chemical qualities; and that it is incorrect to call one inert to oil if it has a noticeable effect on it, whether to retard drying, like gypsum or lampblack, or to hasten the destruction of the film, like $CaCO_3$ or $BaSO_4$. We do not regard spongy platinum, as used in making H_2SO_4, as an inert substance, although it does not dissolve or undergo a chemical change.
>
> All pigments are, by definition, nearly or quite insoluble in oil; PbO and oil form a cement rather than a paint. PbO is considerably soluble in oil; and it is likely (but not certain) that the minute amount of Pb which was found in oil which had been mixed with white lead was from a trace of PbO in the pigment. I can in no other way understand why oil stops dissolving lead when the basic carbonate (containing hydrate) is present in unlimited amount.
>
> In practice we add Pb or Mn compounds to white-lead paint to make it dry, in about the same amount that we do to other paints; for all practical purposes the quantity of Pb dissolved is of no account. It is the same way with red lead; true red lead does not combine with oil in the way Mr. Gardner describes; that with which he experimented probably contained as much as 20 or 30% of PbO. Orange lead, which is red lead nearly or quite free from PbO, will remain uncombined with oil for years, and good ordinary red lead made by modern methods will do so for a month or more.
>
> When saying that white lead is alkaline, Mr. Gardner doubtless means basic; lead is not an alkali metal. If he has any evidence, or knows of any, that white lead or zinc saponify oil at ordinary temperatures, he should publish it; it would be new and important; also how 10% of a chemically inactive substance can hinder saponification.
>
> A. H. Sabin.

The Guide Framing of a Gas-Holder.

By E. F. MILLER.*

DESCRIPTIVE.—A gas-holder, being a bell or inverted bucket supported by the confined gas and floating in water, is in unstable equilibrium because the weight of the crown, or bottom of the bucket, is at the highest point. For this reason, and also for the purpose of resisting the overturning force of the wind, lateral support must be furnished to keep the bottom edge, or curb, approximately level with the surface of the water in the tank. As the gas-holder rises and falls these supports become guides.

The guiding of the bottom curb consists of rails secured to the inside of the shell of the tank, acting as tracks for the rollers secured to the curb of the holder (Fig. 1). The guides for the top curb rollers must extend above the tank, and the structure to maintain them in vertical lines, consisting of standards, struts, ties and necessary details, is called the guide frame.

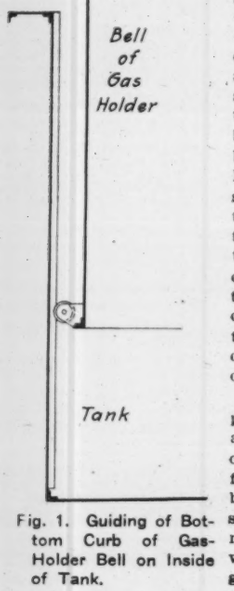


Fig. 1. Guiding of Bottom Curb of Gas-Holder Bell on Inside of Tank.

The base of this guide frame is fastened to the top curb of the tank. In the case of a gasholder in a masonry tank, the standards are fastened to base plates secured by foundation bolts built into the walls. In a steel tank the standards are secured to the top curb of the tank, and also to the tank shell, by means of plates uniting them to the tank standards or column supports, the latter being bolted or riveted to the plates of the tank shell.

The guide frame proper, lying entirely above the tank curb, consists (see Fig. 2) of four kinds of members: 1, the vertical standards V (usually miscalled columns), which carry the guides; 2, the horizontal struts H (usually miscalled girders); 3, the diagonal ties D in each rectangular bay, and 4, the wind ties W at the top, which help to preserve the geometrical figure.

HISTORICAL.—The older gas-holders, consisting of a single lift working in a masonry tank, did not present a very large surface to the wind, so that a number of posts set up about the circle of the tank curb, tied together (sometimes) at the top, and carrying cast-iron guides, were sufficient as lateral supports.

Being designed by architects, these guide frames were usually constructed of cast-iron, in conventional and ornamental shapes. A round column with base and capital and shaft, strictly according to one of the classical orders, was the favorite, although rectangular columns built of cast-iron plates bolted together were frequently used. These plates were cast with exterior moldings, and with openings, all of graceful design, sometimes of Italian scroll work, sometimes of Gothic arches. As it was thought desirable to distribute gas at a pressure much less than the gas-holder gave (unless built very flimsily) counterbalancing was applied, the counter-balance weights usually moving inside of the hollow columns. The so-called girders which sometimes united the tops of the columns, were, when of cast-iron, usually made with a curved upper flange and an openwork web which was designed with circles, fleur-de-lis, etc. Ornamental cast-iron urns were sometimes placed on top of the columns and were filled with earth in which vines could grow.

Neither diagonal ties nor wind ties were used in these designs, and each standard resisted its transverse load by the moment of its own weight and by the resisting moment of its foundation bolts. Although the posts or guide supports received no vertical load other than their own weight and that of the other parts of the guide frame, they were given the architectural name of "columns," which is still more or less used when speaking of the corresponding parts in modern structures; likewise the horizontal members which were used to unite the tops of the columns were called "girders" although they supported no transverse load other than their own weight, and this name is still more or less used.

As the building of telescopic gas-holders came into vogue and more attention was paid to efficiency of construction, round tapered columns were made of wrought-iron plates rolled into sections which were united by inside butt straps having rivets countersunk on the outside; after painting, the appearance of such columns was similar to the round ones of cast-iron. The girders were also made of wrought-iron, generally of the flat type, having angles in the top and bottom chords, these angles being sometimes covered by plates; the chords were connected by flat lattices. Diagonal ties of round iron also began to be used between columns.

Finally, with the advent of multiple-lift gas-holders working in steel tanks, the design of the guide frame was changed to the steel structure, riveted together and having diagonal ties and wind ties, which is now generally adopted.

Assumptions as a Basis for Calculating Loads on Guide Frames.

The forces which a guide frame must resist are due to the pressure and impulses of the wind, to the unbalanced snow load, and to the unbalanced weight of the gas-holder bell. As a gas-holder is designed to be of uniform weight on its circumference, the unbalanced condition of the bell is purely accidental and cannot be estimated; its effect is therefore neglected, excepting in so far as it is covered, together with other uncertainties, by the so-called factor of safety. The snow load, when uniformly distributed over the crown, does not augment the forces which must be resisted by the strength and stiffness of the guide frame, but when accumulated on one side it throws the gas-holder bell out of balance and tends to distort the guide frame. When the snow is on the lee side of the crown it adds a certain amount to the estimated wind load. The usual assumption of unbalanced snow load is 5 lbs. per sq. ft. on one-fourth of the crown area (Fig. 3), the center of gravity of the unbalanced load being assumed to be a point located at two-thirds of the distance from the center of the crown to its outer edge.

The wind is the most powerful distorting force to be considered. The guide frame of a gas-holder which is either fully or partially inflated is not in danger of distortion, provided its members are strong enough to properly resist the direct loads, but a guide frame standing by itself is in great danger of accumulation of loads due to vibration caused by the impulses of the wind, and on this account the details of the joints, as well as the sections of the members, are of great importance. The impulses of the wind are not of large extent or duration, and will not overcome quickly the inertia of a relatively large and heavy gas-holder; further, these impulses may succeed each other with considerable frequency, but it is improbable that they will hit the same spot, or hit from the same direction. But in a guide frame standing alone, a succession of blows may have a cumulative effect on short vibrations, setting up local strains in excess of results of static calculations. When repeated and prolonged, these local excessive strains are more destructive of details than the maximum calculated strains. An efficient design of the wind ties at the top of the guide frame will largely promote the preservation of its figure and shape, and reduce the amplitude of vibrations.

A large guide frame may be considered to be a relatively thin cylinder, secured at the bottom and kept in shape at the top, but near the middle the tendency to change of shape is resisted only by the horizontal struts considered as rings, and the vertical standards considered as beams. The struts being, say, only 30 ins. deep horizontally, and the standards perhaps only 48 ins. deep radially, it is readily seen that the wind-tie system at the top should be deep, sufficient, and efficient.

Many guide frames are built without wind ties, but a sufficient rigidity of the top of the frame can be insured more economically by the use of wind ties than in any other manner. Here it must be remembered that it is impossible to load and test a gas-holder frame as a bridge or floor may be tested, and it is not probable that one in a thousand frames is ever subjected to a load as severe as is assumed in the calculations. For this reason guide frames of defective design may have a long and useful life and be repeated, having the approval of experience.

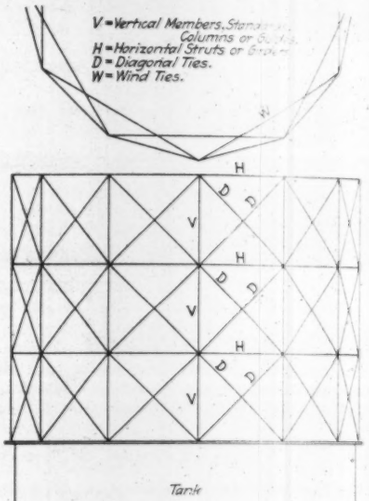


Fig. 2. Outline Diagram of Gas-Holder Guide Frame.

There is a design of shortened frame, permitting one or two sections of the gas-holder to rise above the top. In this design the top strut or girder is made wide, providing a horizontal girder of considerable depth to stiffen the frame and to keep the holder in shape when the wind pressure against the upper section tends to unbalance loads and distort the holder rings.

A horizontal wind load of 32 lbs. per sq. ft. on a normal plane surface, or 16 lbs. on the diametral projection of a cylindrical surface, which corresponds to an observed velocity of about 115 miles per hour (or a corrected velocity of about 90 miles per hour, according to the tables of the U. S. Weather Bureau), exceeds the records at any observing station in this country.*

Calculation of Loads and Stresses.

To avoid the construction and handling of long formulas with many terms, which must be carried along, an example will be taken and the formulas developed along with the actual calculation of loads.

Consider a four-lift gas-holder of about 3,700,000 cu. ft. capacity, of these dimensions:

	Diameter ft.	Diameter ins.	Depth ft.	Depth ins.
Inner lift	181	2	35	0
Second lift	184	1	35	6
Third lift	186	9	35	4
Outer lift	189	5	35	10
Tank	192	0	36	0

Guide Frame: 20 standards, 142 ft. high, with 5 rings of horizontal struts.

*Camden Iron Works, Camden, N. J.

*An exhaustive presentation of existing information about wind velocity and pressures may be found in an article by Captain Ihxby, printed in Engineering News, March 14, 1895. Also see publications of the U. S. Weather Bureau, and published tables of record of observations. The maximum recorded velocity at any city in the United States since the establishment of the Weather Bureau, is 84 miles per hour, observed at Chicago, corresponding to a pressure of 16.9 lbs. per sq. ft.

All experiments indicate that the measured specific pressure of the wind decreases with the increase of the size of the target.

Each lift is guided on the standards by 20 goosenecks or top brackets, each carrying one radial and two tangential rollers. To determine the general loads on the guide frame it is sufficient to calculate the total load at the top of each lift, but to design the section of the standards the maximum load on the individual rollers must be ascertained. This is figured as below.

All rollers standing at an angle of less than 60° with the direction of the wind, and so located that any horizontal movement caused by the wind will be toward the guide, are assumed to be effective and to carry loads proportional to the cosines of the respective angles with the wind direction. The effective resistance of these rollers to horizontal translation (due to their reaction against the guide frame) is also proportional to the cosines of the angles, therefore the relative effective resistance offered is proportional to the squares of the cosines of the respective angles with the wind direction. It is evident that the radial roller directly in line with the wind and the tangential rollers at right angles will carry the greatest loads. Refer to Fig. 4:

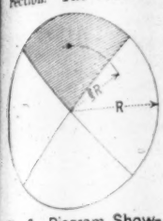


Fig. 3. Diagram Showing Assumed Maximum One-Sided Snow-Load.

Radial	1 cos² 0°	=	1.00
"	2 cos² 18°	=	1.81
"	2 cos² 36°	=	1.31
"	2 cos² 54°	=	.69
Tangential	2 cos² 0°	=	2.00
"	4 cos² 18°	=	3.62
"	4 cos² 36°	=	2.62
"	4 cos² 54°	=	1.38
			14.43

It is thus seen that the total horizontal load at any ring of goosenecks divided by 14.43 will be the maximum roller load, which is also the maximum central load that a section of standard must carry, either radially or tangentially.

The roller pressures are found as follows:

The horizontal load on the goosenecks at the top of the inner lift is:

Due to snow,

$$\frac{\text{Area of crown} \times 5}{4} \times \frac{\text{⅝ of radius}}{\text{½ net depth of four lifts}} = \frac{25{,}850 \times 5 \times 60.5}{4 \times 68.6} = 28{,}500 \text{ lbs.}$$

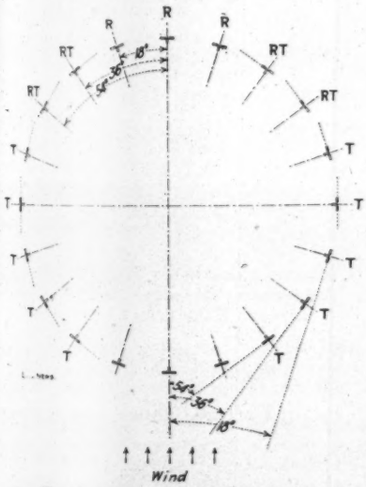


Fig. 4. Diagram of Guide Rollers.

Due to wind,

Diam. of inner lift × 16 × ½ net depth of inner lift = 181.4 × 16 × 16.7 = 48,500 lbs.

Total horizontal load on goosenecks = 77,000 lbs.

The maximum load on individual rollers

at the top of the inner lift is $\frac{77{,}000}{14.43} =$ 5,300 lbs.

The horizontal load on the goosenecks at top of second lift is,

Diam. of inner lift × 16 × ½ net depth of inner lift + Diam. of second lift × 16 × ½ net depth of second lift = 48,500 + (184.1 × 16 × 17) = 48,500 + 30,100 = 98,600 lbs.

The maximum load on individual rollers at top of the second lift is $\frac{98{,}600}{14.43} =$ 6,860 lbs.

The horizontal load on the goosenecks at top of third lift is 50,100 + 186.8 × 16 × 17 = 50,100 + 50,800 = 100,900 lbs.

The maximum load on individual rollers at the top of the third lift is $\frac{100{,}900}{14.43} =$ 6,900 lbs.

The horizontal load on the goosenecks at top of outer lift is 50,800 + 189.4 × 16 × 17.9 = 50,800 + 54,200 = 105,000 lbs.

The maximum load on individual rollers at top of outer lift = $\frac{105{,}000}{14.43} =$ 7,300 lbs.

In transferring the figured loads from the goosenecks to the guide frame, the front flange of the standard is considered as carrying the maximum tangential roller-load as a beam, loaded at the center.

A standard 15-in. I-beam 28 ft. long, weighing 42 lbs. per ft., is stressed to a maximum of 16,000 lbs. per sq. in. by a center load of 11,200 lbs. The maximum figured load of 6,900 lbs. (third lift) is sufficiently inside this figure to produce a maximum stress of 9,900 lbs. per sq. in. The figured deflection will be about 0.43-in. As the I-beams are firmly united at their ends by flanged butt-straps, thus forming continuous beams, the stress and deflection are probably much less than the figures indicate.

In order to ascertain the maximum stresses in the diagonal ties, horizontal struts and the back flanges of the standards, the gooseneck loads must be distributed to the panel points; this distribution is made by considering the sections of standards as beams (supported at the panel points) upon which the loads act. In addition to the loads from the goosenecks there are further slight load due to the direct pressure against the standards, which present surfaces (assumed to be 4 ft. wide) normal to the wind. The sum of both loads will give the maximum horizontal shear, which is considered as being resisted by the diagonal ties. Refer to Fig. 5.

Load distributed to top of 1st (upper) bay:—

Horizontal load from goosenecks = 77,000 lbs.

Horizontal load from wind on standards

$$2 \times \frac{\text{length 1st section} \times \text{width}}{2} \times 32 = 2 \times \frac{30 \times 4}{2} \times 32 = \quad 3{,}800 \text{ lbs.}$$

Total maximum horizontal shear = 80,800 lbs.

Load distributed to top of 2d bay:

Maximum horizontal shear from above = 80,800 lbs.

Horizontal load from goosenecks = $\frac{98{,}600 \times 264}{336} =$ 77,500 lbs.

Horizontal load from wind on standards =

$$2 \times \frac{\text{length first section} \times \text{width}}{2} \times 32 + 2 \times \frac{\text{length second section} \times \text{width}}{2} \times 32 =$$

$3{,}800 + 2 \times \frac{28 \times 4}{2} \times 32 = 3{,}800 + 3{,}600 =$ 7,400 lbs.

Total maximum horizontal shear = 165,700 lbs.

Load distributed to top of 3rd bay:—

Maximum horizontal shear from above = 165,700 + 98,600 — 77,500 = 186,800 lbs.

Horizontal load from goosenecks = $\frac{100{,}900 \times 178}{336} =$ 53,500 "

Horizontal load from wind on standards = $3{,}600 + 2 \times \frac{28 \times 4}{2} \times 32 =$ 7,200 "

Total maximum horizontal shear = 247,500 lbs.

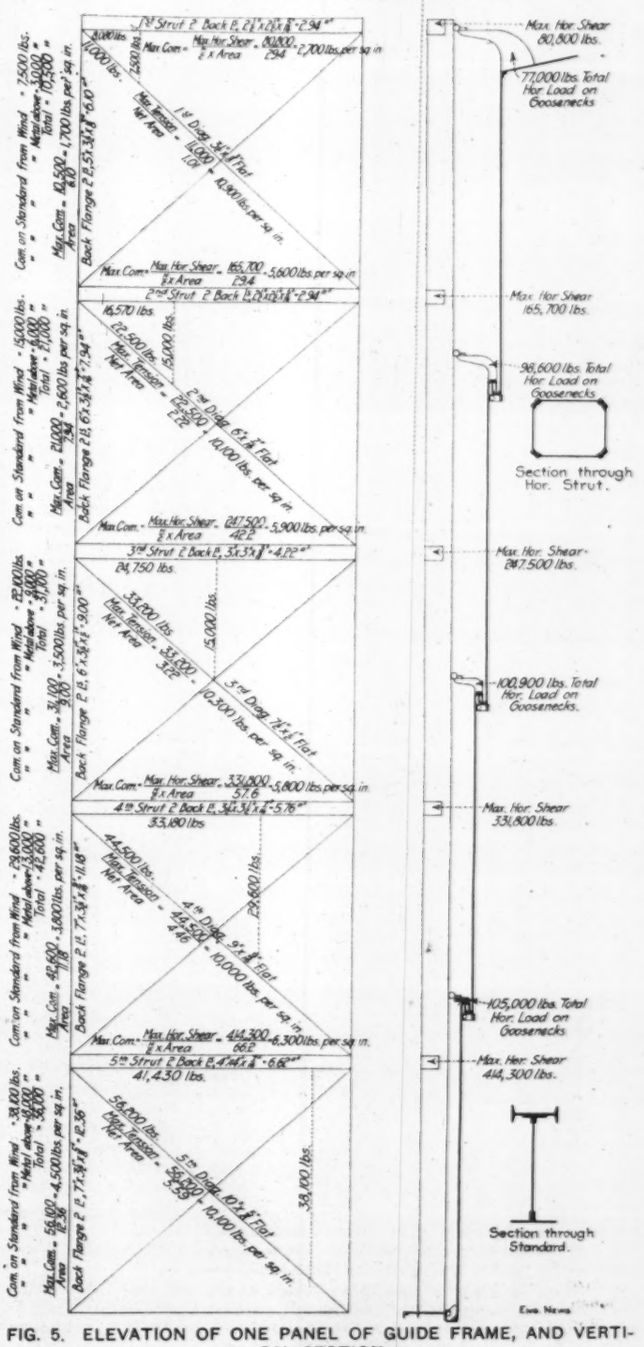

1st Strut 2 Back B, 6½"x12"x ⅜" = 2.94"
Max. Com. = Max. Hor. Shear / 8 x Area = 80,800 / 294 = 2,700 lbs. per sq. in.
Com. on Standard from Wind = 7,500 lbs.
" " " Metal above = 3,000 "
" " " Total = 10,500 "
Max. Tension = 10,500 / Net Area = 1,700 lbs. per sq. in.
Max. Com. = 10,500 / Area = 1,700 lbs. per sq. in.
Back Flange 2 B, 5½"x½" = 6.00"
1st Diag. 3½"x ⅜" Flat
10,000 lbs. per sq. in.
Max. Com. = Max. Hor. Shear / 8 x Area = 165,700 / 294 = 5,600 lbs. per sq. in.
2nd Strut 2 Back B, 6½"x½" = 7.94"
16,570 lbs.
Com. on Standard from Wind = 15,000 lbs.
" " " Metal above = 6,000 "
" " " Total = 21,000 "
Max. Com. = 21,000 / 7.94 = 2,600 lbs. per sq. in.
Back Flange 2 B, 6½"x½" = 7.94"
2nd Diag. 6½"x ⅜" Flat
10,000 lbs. per sq. in.
Max. Com. = Max. Hor. Shear / 8 x Area = 247,500 / 42.2 = 5,900 lbs. per sq. in.
3rd Strut 2 Back B, 3½"x½" = 4.22"
24,750 lbs.
Com. on Standard from Wind = 22,100 lbs.
" " " Metal above = 9,000 "
" " " Total = 31,100 "
Max. Com. = 31,100 / 9.00 = 3,500 lbs. per sq. in.
Back Flange 2 B, 6½"x½" = 9.00"
3rd Diag. 7½"x ⅜" Flat
10,300 lbs. per sq. in.
Max. Com. = Max. Hor. Shear / 8 x Area = 331,800 / 57.6 = 5,800 lbs. per sq. in.
4th Strut 2 Back B, 3½"x½" = 5.76"
33,180 lbs.
Com. on Standard from Wind = 29,600 lbs.
" " " Metal above = 13,000 "
" " " Total = 42,600 "
Max. Com. = 42,600 / 11.18 = 3,800 lbs. per sq. in.
Back Flange 2 B, 7½"x½" = 11.18"
4th Diag. 9½"x ⅜" Flat
10,000 lbs. per sq. in.
Max. Com. = Max. Hor. Shear / 8 x Area = 414,300 / 66.2 = 6,300 lbs. per sq. in.
5th Strut 2 Back B, 4"x6½" = 6.62"
41,430 lbs.
Com. on Standard from Wind = 38,100 lbs.
" " " Metal above = 20,000 "
" " " Total = 58,100 "
Max. Com. = 58,100 / 12.36 = 4,500 lbs. per sq. in.
Back Flange 2 B, 7½"x½" = 12.36"
5th Diag. 10½"x ⅜" Flat
10,100 lbs. per sq. in.
Max. Hor. Shear 60,800 lbs.
77,000 lbs. Total Hor. Load on Goosenecks
Max. Hor. Shear 165,700 lbs.
98,600 lbs. Total Hor. Load on Goosenecks
Section through Hor. Strut.
Max. Hor. Shear 247,500 lbs.
100,900 lbs. Total Hor. Load on Goosenecks.
Max. Hor. Shear 331,800 lbs.
105,000 lbs. Total Hor. Load on Goosenecks.
Max. Hor. Shear 414,300 lbs.
Section through Standard.
Eng. News

FIG. 5. ELEVATION OF ONE PANEL OF GUIDE FRAME, AND VERTICAL SECTION.

Load distributed to top of 4th bay:—
Maximum horizontal shear from above = 247,500 + 100,900 — 53,500 = 294,900 lbs.
Horizontal load from goosenecks = $\frac{105{,}000 \times 95}{336}$ = 29,700 "
Horizontal load from wind on standards = $3{,}600 + 2 \times \frac{28 \times 4}{2} \times 32$ = 7,200 "
Total maximum horizontal shear = 331,800 lbs.

Load distributed to top of 5th (lower) bay:—
Maximum horizontal shear from above = 331,800 + 105,000 — 29,700 = 407,100 lbs.
Horizontal load from wind on standards = $3{,}600 + 2 \times \frac{28 \times 4}{2} \times 32$ = 7,200 "
Total maximum horizontal shear = 414,300 lbs.

The guide frame acts as a whole, similarly to a hollow cylinder fixed at its lower end and loaded as a cantilever beam. One diagonal tie in each bay (excepting those which may happen to stand in planes normal to the wind) is active as a tension member. The horizontal component of the

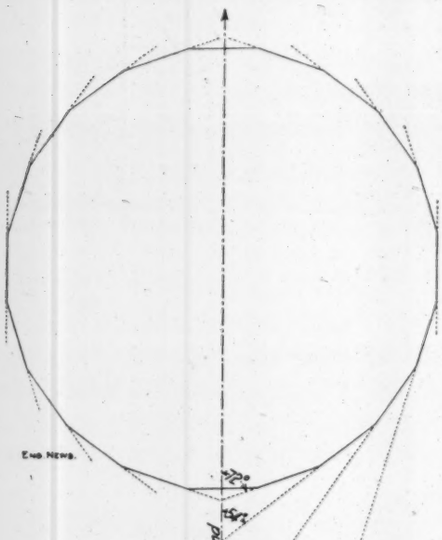


Fig. 6. Diagram of Bracing Panels.

load which each diagonal tie receives is proportional to the cosine of the angle of its plane with the direction of the wind, but its resistance in a direction parallel to the wind is also proportional to the cosine of the angle; therefore the relative effective resistance of the diagonal tie to the horizontal shear is proportional to the square of the cosine of the horizontal angle. It is evident that the diagonal ties which stand in planes parallel to the wind will receive the greatest loads. Refer to Fig. 6.

2	$\cos^2$	0°	= 2.000
4	$\cos^2$	18°	= 3.618
4	$\cos^2$	36°	= 2.618
4	$\cos^2$	54°	= 1.382
4	$\cos^2$	72°	= .382
			10.000 = ½ N

It is thus seen that ½ N, or half the total number of standards, divided into the total maximum horizontal shear will give the horizontal component of the greatest load on any diagonal tie.

Horizontal component of maximum load on 1st diagonal tie:

$$\frac{80{,}800}{10} = 8{,}080 \text{ lbs.}$$

Horizontal component of maximum load on 2d diagonal tie:

$$\frac{167{,}500}{10} = 16{,}750 \text{ lbs.}$$

Horizontal component of maximum load on 3d diagonal tie:

$$\frac{247{,}500}{10} = 24{,}750 \text{ lbs.}$$

Horizontal component of maximum load on 4th diagonal tie:

$$\frac{331{,}800}{10} = 33{,}180 \text{ lbs.}$$

Horizontal component of maximum load on 5th diagonal tie:

$$\frac{414{,}300}{10} = 41{,}430 \text{ lbs.}$$

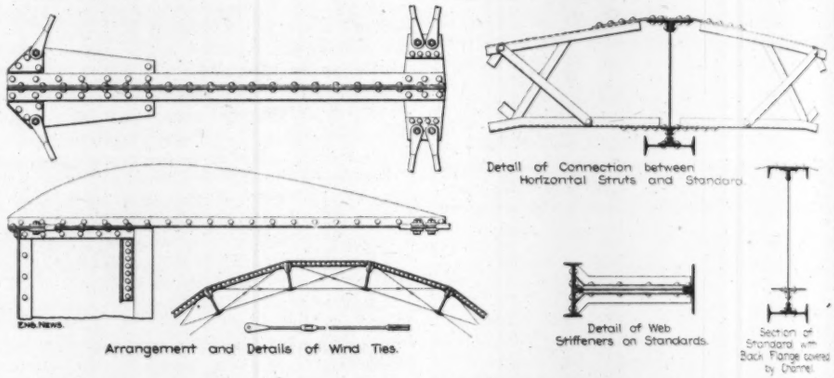
The direct load thrown on each diagonal tie is the resultant of the horizontal and vertical components as determined by its vertical angle. These resultants are readily ascertained graphically; they are given in Fig. 5. The sections selected and the resulting unit-stresses in the diagonals are:

1st bay, 3½ × ⅜-in. flat,

$$\frac{\text{Maximum tension}}{\text{Net area}} = \frac{11{,}000}{1.01} = 10{,}900 \text{ lbs. per sq. in.}$$

2d bay, 6 × 7/16-in. flat,

$$\frac{\text{Maximum tension}}{\text{Net area}} = \frac{22{,}500}{2.22} = 10{,}100 \text{ lbs per sq. in.}$$


Eng. News.
Detail of Connection between Horizontal Struts and Standard.
Detail of Web Stiffeners on Standards.
Section of Standard with Back Flange cover by Channel.

Arrangement and Details of Wind Ties.

FIG. 7. DETAILS OF WIND TIES AND CONNECTION OF STRUTS TO STANDARDS.

3d bay, 7½ × ½-in. flat,

$$\frac{\text{Maximum tension}}{\text{Net area}} = \frac{33{,}200}{3.22} = 10{,}300 \text{ lbs. per sq. in.}$$

4th bay, 9 × 9/16-in. flat,

$$\frac{\text{Maximum tension}}{\text{Net area}} = \frac{44{,}500}{4.46} = 10{,}000 \text{ lbs. per sq. in.}$$

5th bay, 10 × ⅝-in. flat,

$$\frac{\text{Maximum tension}}{\text{Net area}} = \frac{56{,}200}{5.59} = 10{,}100 \text{ lbs. per sq. in.}$$

The maximum load on any horizontal strut is the horizontal component of the maximum load on the diagonal tie lying in the same bay, which is equivalent to the total maximum horizontal shear divided by ½ N. The sections adopted and the resulting unit-stresses are:

1st bay, two back angles 2½ × 2½ × 5/16-in.,

$$\frac{\text{Total maximum horizontal shear}}{\text{½ } N \times \text{area of two back angles}} = \frac{80{,}800}{10 \times 2.94} = 2{,}700 \text{ lbs. per sq. in.}$$

2d bay, two back angles 2½ × 2½ × 5/16-in.,

$$\frac{\text{Total maximum horizontal shear}}{\text{½ } N \times \text{area of two back angles}} = \frac{165{,}700}{10 \times 2.94} = 5{,}600 \text{ lbs. per sq. in.}$$

3d bay, two back angles 3 × 3 × ⅜-in.

$$\frac{\text{Total maximum horizontal shear}}{\text{½ } N \times \text{area of two back angles}} = \frac{247{,}500}{10 \times 4.22} = 5{,}900 \text{ lbs. per sq. in.}$$

4th bay, two back angles 3½ × [illegible] × 7/16-in.,

Total maximum horizontal shear [illegible]

$$\frac{\text{Total maximum horizontal shear}}{\text{½ } N \times \text{area of two back angles}} = \frac{331{,}800}{10 \times 5.76} = 5{,}8[illegible] \text{ lbs. per sq. in.}$$

5th bay, two back angles 4 × 4 × [illegible] × 7/16-in.,

Total maximum horizontal shear 414,300

$$\frac{\text{Total maximum horizontal shear}}{\text{½ } N \times \text{area of two back angles}} = \frac{414{,}300}{10 \times 6.02} = 6{,}3[illegible] \text{ lbs. per sq. in.}$$

The struts are square and consist of four angles, one in each corner, connected by lattice-bars on all four sides. The least radius of gyration of the strut will be that of the combined section, although the loads are considered as being distributed over the back angles only. The maximum compressions (per square inch of area in back angles only) for the various struts are well within the allowable unit-stress limits which correspond to the unsupported lengths divided by the least radii of gyration, as given in handbooks.

When considering the standards as beams loaded radially, the maximum stresses will occur in the standard farthest from the center on the lee side, due to the maximum radial roller load. The maximum stresses per square inch will occur in the second section (28 ft. long), the standard being considered as a non-continuous beam loaded in the middle and supported at the ends.

The front flange consists of a 15-in. 42-lb. I-beam and two 3½ × 2½ × 5/16-in. angles, the gross area of the three shapes being 16.04 sq. ins. The back flange consists of two 6 × 3½ × 7/16-in. angles, the gross area of the two shapes being 7.94 sq. ins. and the net area 7.12 sq. ins.

The maximum bending moment is $\frac{6{,}800 \times 336}{4}$ = 571,200 in.-lbs. The distance from the centers of gravity of the front and back flanges to the neutral axis of the beam are 11.59 ins. and 23.41 ins. respectively. Then

$$23.41 \times 7.94 \times T + 11.59 \times 16.04 \times C = 571{,}200.$$

In the above equation T and C are the tension and compression, per square inch of gross area, in the back and front flanges respectively.

But $C = \frac{11.59}{23.41} T$, and by substitution we have

$$23.41 \times 7.94\ T + 11.59 \times 16.04 \times \frac{11.59}{23.41} T = 571{,}200.$$

Whence

$$277.9\ T = 571{,}200,$$

or

$$T = 2{,}055 \text{ lbs. and } C = 1{,}017 \text{ lbs.}$$

But T is to be increased in the proportion of $\frac{7.94}{7.12}$ when net area only is considered. Therefore the fiber stress (tension) in the back flange is 2,300 lbs. per sq. in. net, that in the front flange (compression) remaining 1,0[illegible] lbs. per sq. in. gross.

The greatest stresses due to the loads from the tangential rollers will occur in the standards at

right angles to the direction of the wind, which fiber stresses have previously been shown to be 9,900 lbs. per sq. in.

The greatest direct compression on a standard is due to the vertical component of the maximum load on the diagonal tie in the same bay, plus the weight of the metal in the structure above. This compression will take place in the same standard where the maximum load from the tangential roller occurs, but the compression due to the load from the diagonal tie is considered as taken by the back flange only, while the load from the tangential roller is considered as taken by the front flange only. The vertical components of the loads on the diagonal ties are readily ascertained graphically. They are given in Fig. 5. The stress computation for the back flanges of the standards is as follows:

1st bay, two back angles 5 × 3½ × ⅜-in.,

$$\frac{\text{Vertical component + weight of metal above}}{\text{area of two back angles}} = \frac{7{,}500 + 3{,}000}{6.1} = 1{,}700 \text{ lbs. per sq. in.}$$

2d bay, two back angles 6 × 3½ × 7/16-in.,

$$\frac{15{,}000 + 6{,}000}{7.94} = 2{,}600 \text{ lbs. per sq. in.}$$

3d bay, two back angles 6 × 3½ × ½-in.,

$$\frac{22{,}100 + 9{,}000}{9} = 3{,}500 \text{ lbs. per sq. in.}$$

4th bay, two back angles 7 × 3½ × 9/16-in.,

$$\frac{29{,}600 + 13{,}000}{11.18} = 3{,}800 \text{ lbs. per sq. in.}$$

5th bay, two back angles 7 × 3½ × ⅝-in.,

$$\frac{38{,}100 + 18{,}000}{12.36} = 4{,}500 \text{ lbs. per sq. in.}$$

The least radius of gyration for any section of standard is that of the combined section of the back flange. The maximum compressions (per sq. in. of area in back flanges only) for the various sections are well within the allowable unit-stress limits which correspond to the unsupported lengths divided by the least radii of gyration, as given in hand-books.

It is not possible to accurately calculate the stresses in the wind ties, but their dimensions are determined by judgment and experience. In the case under consideration the ties consist of rods of 1¼-in. diameter, having the ends upset for 1½-in. turnbuckles for adjustment. The arrangement is such (Fig. 7) that the effective depth of the wind tie system, considered as a stiffening ring, is about 10 ft.

In determining the loads at the bases of the standards, due to the overturning moment caused by the wind pressure, the guide frame is considered as a cylinder having a diameter of 194 ft. 6 ins. and a height of 142 ft., uniformly loaded at 16 lbs. per sq. ft. on the diametral projection. The total wind moment will then be 194.5 × 142 × 16 × 71 = 31,375,200 ft.-lbs. The loads at the bases of the various standards are proportional to their distances from the neutral axis, those lying farthest away receiving the greatest loads and those lying on the neutral axis receiving none. The effect of the resistance of each standard to the wind moment is also proportional to its distance from the neutral axis; therefore the relative effective resistance offered to the wind moment is proportional to the square of its distance from the neutral axis. Refer to Fig. 8.

$$\begin{aligned}
2 \cos^2 0^\circ \; R &= 2.000 \; R \\
4 \cos^2 18^\circ \; R &= 3.618 \; R \\
4 \cos^2 36^\circ \; R &= 2.618 \; R \\
4 \cos^2 54^\circ \; R &= 1.382 \; R \\
4 \cos^2 72^\circ \; R &= .382 \; R \\
&\overline{10.000 \; R}
\end{aligned}$$

It is thus seen that the total wind moment, divided by the product of half the number of standards into the radius to the center of the standard, will give the maximum load at the base of any standard.

$$\frac{31{,}375{,}200}{10 \times 97.25} = 32{,}300 \text{ lbs.}$$

The loads in those standards on the lee side will produce pressure on the base, while on the windward side a lift will occur. Therefore the maximum pressure due to the wind moment and the load from the weight of metal in the structure above will be 32,300 + 18,000 = 50,300 lbs., while the maximum lift will be 32,300 — 18,000 = 14,300 lbs.

General Remarks.

It has sometimes been contended that the standard supports, which are attached to the tank shell and located directly under the guide-frame standards, should extend to the tank foundation and rest thereon in order to properly support the guide frame, but in view of the light loads actually existing at the bases of the standards this is obviously unnecessary. It is usually sufficient that these supports extend to about one-half the depth of the tank. The loads from the guide frame are taken up by the rivets which attach the supports to the tank shell, and there should always be a sufficient number of these to keep the shear and bearing much below the allowable limits, which is easily accomplished without extending to the bottom.

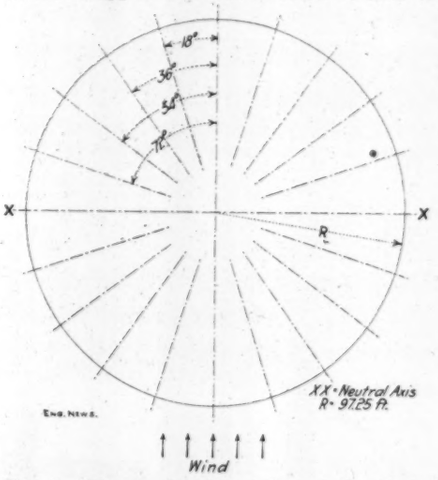


Fig. 8. Diagram for Calculation of Overturning Effect.

In order to secure rigidity of the guide frame, all joints between members should be made with rivets, with the single exception of the wind ties, where bolts and turnbuckles are permissible to accomplish adjustment.

In especially high guide frames the tension in the lower diagonals should be kept within say 12,000 lbs. per sq. in., in order to avoid such stretch as would allow of undesirable deflection at the top of the guide frame.

In addition to having sufficient strength to resist the loads applied by the tangential rollers, the front flange of the standard should be of liberal width in order to limit deflection.

The radial loads being comparatively small, only two sets of stiffeners (each set consisting of one angle on each side) are applied to the web of each section of standard. The use of these stiffeners to square the flanges when fitting up in the shop, and to keep the standard in shape, is considered to be of more importance than their function as web stiffeners.

In no case should loads be carried by rivets in tension. When designing connections between struts and standards, or between diagonals and standards, it is necessary, in order to avoid subjecting rivets to tension, that any attachment to the web of the standard be made by means of a gusset-plate which is continuous through the web, thus allowing of connecting the ends of struts or diagonals coming on opposite sides of the web by rivets in shear through the gusset plate.

In designing joints at the ends of a diagonal, sufficient rivets should be used to take up at 10,000 lbs. per sq. in. shear a load which could be produced by a stress of 16,000 lbs. per sq. in. on the net section of the diagonal. All other joints, especially at the panel points, should also be provided with an excess of rivets, the object being to have an abundance in case some should be loosened by racking and vibrations due to the impulses of the wind. This excess of rivets also serves to keep the shear and bearing well within the allowable limits used in approved bridge construction.

Eccentric loading is avoided as much as possible and where it does occur is of such slight extent as to be abundantly cared for by the so-called factor of safety. The center line of riveted joints should be made to coincide with the center of gravity of the section, and this is easily accomplished in most cases, for instance, when the diagonals are flats; but when they are angles it necessitates that rivets be put through both legs, using a knee on the leg which is perpendicular to the gusset plate.

As the loads which must be resisted by a guide frame may be applied from any direction, it follows that all members of the same kind which are situated in the same section of the guide frame should be of identical design. For this reason only those calculations are made which are necessary to determine maximum stresses. It is to be observed in the foregoing calculations that the entire snow load is assumed as being taken by the goosenecks at the top of the inner lift. Actually, part of this load will be taken by other goosenecks than those on the inner lift, but to determine how much would be impossible because the rigidity of the gas-holder bell—an uncertain quantity—would be involved. Therefore the above assumption is made, as it gives loads which are greater than those which would result from any other assumption.

Reversal of stresses will occur in some of the members of a guide frame because the loads may be applied from any direction, but as in the most approved practice the stresses are kept within, say, 12,500 lbs. per sq. in. when resisting (in addition to snow loads) loads resulting from a wind pressure of 32 lbs. per sq. ft. against a plane surface, and as storms violent enough to produce such a pressure will probably occur only once in a term of years, it is obvious that the matter of such reversal is of so little importance that it may be disregarded and considered as being covered by the so-called factor of safety.

In making calculations for actual designs it will usually be best to consider the diametral projection, on which the wind acts, as having a horizontal dimension equal to the diameter at the outermost parts of the standards, because this assumption will provide for all additional loads which may occur due to the passing of the wind between the gas-holder bell and the guide frame, and its possible consequent entanglement with, and pressure upon, the various members.

When the flange of the standard is supported (opposite the panel points) by the struts, which have their front angles set near the ends and attached to the web as near the front flange as possible in the manner indicated in Fig. 7, and when the standards are otherwise made strong enough to carry to the back flange the loads transmitted by the gooseneck rollers, it is best that the diagonals should be located at the back flange. When they are so located the connection between the diagonals, struts and standard is made by a single gusset-plate having all its rivets in direct shear, and all the loads transmitted by the gooseneck rollers are thus finally carried through the gusset-plates at the panel points and are there directly distributed to the various members which resist these loads. Furthermore, all stresses which are not finally neutralized by the weight of the guide frame are transmitted to the standard supports by gusset-plates on the back flanges, the rivets being in direct shear. When the diagonals are placed close to the front flange it is necessary, in order to secure clearance between them and the gas-holder, to pro-

vide a greater number of standards than would be required if the diagonals were located at the back flange, which is an advantage in favor of the latter construction. Another advantage is that the tension in the back flanges, produced by the radial roller loads, is either wholly or partly neutralized by the compression resulting from the pull of the diagonals, whereas the compression in the front flanges, produced by the radial and tangential roller loads, would be augmented by placing the diagonals near the front flange.

The use of box-shaped rather than I-shaped horizontal struts is advisable in large guide frames because, for the same weight of metal used, the least radius of gyration obtained in the box construction is much greater than in the I design.

The choice of either latticed or web-plate for use as diagonals is dependent upon circumstances. Flats are cheapest in first cost because the price of the metal in them is generally less per pound than for the alternate shapes, and also because the shop and field work is less expensive. When either angles or channels are used it is necessary, in order to properly distribute the stresses about the center of gravity of the section, to use angle knees on the flanges. When the diagonals are extraordinarily long it is advisable to use angles in order to prevent undue swaying in the wind.

The difficulty of keeping a guide frame properly painted, with the consequent tendency to corrosion, makes it advisable to use no metal in its construction thinner than ¼-in., while for the more important members such as the diagonals and back angles in the struts and standards, the metal should never be thinner than 5/16-in. in large guide frames.

Of these various methods, the wire-rope design is the cheapest to construct, but on account of corrosion, breakage, damage by ice, necessity for constant inspection and repair, and serious consequences of a small failure, it has met with small favor from the engineers of gas companies. Its defects are not serious with small single-lift holders, but the economy of construction for such holders is very little. For multiple-lift holders the economy and defects increase, the defects predominating.

The spiral-guiding design is economical of material, but involves a considerable increase of expense per pound for fabrication and erection, and requires great accuracy of fitting. For multiple-lift holders the presence of numerous sliding brackets and rollers introduces an element of danger in case of a breakage or jam, which does not exist to such an extent with rectangular guides. The design of frame mentioned under 3, having intermediate guides, is a design worked out with great mathematical precision, but certain details are difficult to arrange satisfactorily, and this difficulty increases with the size of the holder. In the nature of the case, the column presents the readiest and most rigid basis for a guiding having a certain inherent stiffness. To ignore this fact is not economical design.

Some designers consider that since radial stiffness is necessarily greater than tangential stiffness, therefore the radial and tangential rollers must not be used together. By the use of liberal tangential guides, the difference may be brought within reasonable limits, and as these structures are not supposed to be loaded to destruction, there is no practical observable difference. When the rollers are at or near the panel points, the radial and tangential guiding is equally efficient.

FIGS. 9 TO 13. VIEWS DURING THE ERECTION OF A LARGE GAS-HOLDER.

standards is made according to considerations which appeal most to the purchaser. The latticed design is cheaper in first cost and is, in our judgment, fully as cheap in maintenance. Many purchasers prefer the web-plate design, thinking that it needs painting less often and can be painted more easily. When the lattices are from ⅜-in. to ½-in. thick the space of ¾-in. to 1-in. between the flange angles is sufficient for the insertion of a paint brush, and, with proper inspection, the structure can be thoroughly painted. Lattices of such thickness will not lose, proportionately, so much from corrosion as web-plates ¼-in. or 5/16-in. thick, and any thicker web-plate increases the first cost considerably. Furthermore, corrosion of flange angles on the surface next to the web-plate is not unknown in plate-girder construction, and when once started the surface cannot be reached for cleaning and painting.

The choice of either flats, channels, or angles

Types of Gas-Holder Guide Frames.

The various methods of guiding gas holders may be classified as follows:

1. Steel frames braced and supplied with tension diagonals such as we are now considering.
2. Steel frames with diagonals which act alternately as ties and struts.
3. Steel frames with guides for tangential rollers only; these guides not on the standards but placed intermediate and supported by the horizontal struts and diagonal braces.
4. Steel frames of various design by firms in France, England and Germany, some having secondary diagonal bracing, others with one ring of very heavy horizontal struts.
5. Wire-rope guiding, designed in England.
6. Spiral guiding, constructed by various firms in England and Germany.

There is a practical advantage in construction, leveling, etc., by use of radial rollers. Furthermore, by the use of both radial and tangential rollers on large holders the number of points of guiding is increased, reducing the specific loads on pins, rollers, guides and fastenings. The general elasticity of the holder and frame will also tend to reduce inequalities. The bearing of a tangential roller on the inside or outside of the flange of a channel, on the flange of a Z-bar, or on the side of the head of a railroad rail, is not to be recommended.

The patented frame with stiff diagonals and no horizontals, presents a pleasing appearance, and undoubtedly is a stiff and rigid design. A long strut, inclined, may, however, be allowed only say 3,000 lbs. strain in compression, while a tension bar may be allowed from 12,000 to 16,000 lbs. tension. Two tension bars will then weigh only about one-half of one compression member, to say nothing of their more simple connections, and less cost of fabrication, being a single rolled section.

The general design presented here has been developed as being economical in material and fabrication and erection, giving satisfactory economical results at least expense.

In conclusion it may well be said that a guide frame, when carefully designed according to the most approved practice and constructed of suitable open-hearth steel, is to be classed among the safest examples of modern structural steelwork.

The Design of Gasholder Tanks.*

By GEORGE EVETTS,† Assoc. M. Inst. C. E.

Gasholder tanks are of two main types: (1) Those mostly or wholly above ground level, usually of steel, but cast iron or reinforced concrete may be employed. (2) Those below ground, of brick (and puddle behind), concrete rendered, a combination of both, of cast iron, or of reinforced concrete. The reinforced-concrete tank is, of course, the latest development in this branch of the work, while the cast-iron tank is now very seldom erected.

For very large holders, assuming good foundations with little water, a brick or concrete tank is most suitable. Some years ago brick or cast-iron tanks were almost universal, but the cheapening of cement led to the introduction of concrete tanks. With ballast at a reasonable price, such tanks are cheaper. Concrete and brick tanks must be puddled at the back, or rendered, to make them watertight. The puddle should be carefully rammed and watered in sections as the wall is built up. Most concrete tanks are rendered, and in order that the rendering make the tank watertight it should be applied when the tank is under tension.

Unless the holder is upwards of 2,000,000 cu. ft. capacity, and the foundations are good, a steel tank is undoubtedly the cheapest form. This type has come into general use for all but the very largest holders since the introduction and cheapening of large rolled plates. They have been constructed for holders up to 5,000,000 cu. ft. capacity, of a diameter up to 240 ft. and a depth of 40 ft.

It should be noted that the rivets in gasholder and tank work have to fulfil a different duty to those in the guide framing and in ordinary constructional work. They have simply to keep the plates together, forming a gas or water-tight joint. In recent tests it has been proved that the ultimate resistance of tank plates is less than the stress that would be necessary to overcome the friction at the joints, and that the friction is so great that within the limits of working stress there is neither shear in the rivet nor bearing stress against the shank. This, of course, applies to first-class riveting, and the rivets should in all cases fit the hole tightly.

The thickness of the bottom row of side plates is easily worked out by the usual formula for boiler-plate thicknesses, in which the pressure is that equal to a head of water equal to the depth of the tank.

For a tank 140 ft. in diameter and 30 ft. deep (suitable for a 1,500,000 4-lift holder) the thickness would be 1 in. The tank would be made of six or seven rows of varying thicknesses, from ⅜-in. at the top. The bottom plates would be ½-in. for outside row about 4 ft. 6 ins. broad, the remaining rows ⅜-in. The 1-in. bottom side plates would be joined to the ½-in. outer row of bottom plates by an internal circumferential angle 6 × 6 × ⅞-in. thick, the rivets would be 1⅛ ins. in diameter, about 3-in. diagonally pitched.. There would be a top circumferential angle and an intermediate one, both fixed to the vertical standards by suitable gussets.

CONSTRUCTION.—The modern type of gasholder sometimes known as the "balloon," consists of a number of comparatively light columns, spaced usually from 15 to 25 ft. apart, securely braced together by horizontal girders and diagonal ties of round or flat bars, and is very economical and reliable. The stresses in any holder and framing are very complex, and each type has a different set of conditions.

The "balloon" type already described is preferable to that of the old massive cast-iron columns with heavy cross girders and very long bracing rods. These heavy columns, however, have an advantage, inasmuch as the columns are well able to preserve the rigid guiding of the holder.

The following is given as an indication of the method of gasholder design:

The wind pressure is taken at from 16 to 20 lbs. per sq. ft. on the diametrical section. The columns are subject to a cantilever stress, the pressure being transmitted on one side directly by the rollers attached to the holder at the corresponding points. Radial rollers only transmit pressure to a quarter of the number of columns, and tangential rollers to a half, but the bracing transmits this pressure through the whole structure.

The horizontal girders are the struts and the diagonal bars are the ties. The whole guide framing therefore can be treated as a huge cylinder stiffened by a thin internal ring (the holder itself). The tendency to buckle is overcome by the stiffness of the holder curbs, cups and vertical stiffeners, in fact, each lift must be rigid in itself without assistance from the framing.

The total capsizing force tends: (1) To break the holder off at the base and overturn it, and (2) to distort the framing out of its cylindrical form.

The whole cantilever therefore has to overcome only the first of these tendencies, in addition to which each cantilever has to overcome the bending moment, due to the wind in each bay. The total vertical shear in each cantilever is then resolved along the other parts of the framing, horizontally for the struts and diagonally for the ties, as in the case of a cantilever-braced girder, whose maximum shear can be found.

Certain portions have to be designed separately. Thus, the top curb, i. e., where the outside row of crown sheets meets the top row of side sheets, is entirely in compression, the amount of which can be approximately calculated. Further, the curved curbs and cups are subject to bending between the columns or points of support and should therefore be sufficiently stiff to avoid springing.

For small holders up to 750,000 cu. ft., the author has found the most economical design to consist of plain I-beams for the vertical standards, 15 to 18 ft. c. to c.; smaller joists for the struts, and circular rods with adjusting screws for the ties. The weights of a 750,000 holder would be:

Tank, inlet and outlet pipes	210 tons	
Guide framing	115 "	(of which diagonal braces weigh only 15 tons)
Three-lift holder	165 "	
Total	490 tons	

For larger holders the lattice-braced vertical and horizontal girders form the cheapest method of construction, but the maintenance costs are more. Generally speaking, the best holder, both from the point of view of initial cost, maintenance and reliability, is the simplest that can be designed without waste of metal.

The rollers should be large and capable of adjustment. Any adjusting bolts of the top carriages should be accessible, and should be independent of the gasholder sheets.

The cups carrying the water for sealing the lift, and its grip, which is an inverted counterpart, should be of the channel type with a stiffening bead at the ends, as shown below. The semi-circular type is bad, as there is initial stress caused by the bending, and a grinding action takes place as the cup and grip meet, if one part is not exactly central to the other. A circumferential channel is a good form for the bottom curb, the plates carrying the bottom rollers being fixed to it.

The sheeting of the crown is from 14 gage to 10, depending on the size of the holders, with thicker plates at the outside and centers. The side plates are 12 gage to 10, with thicker plates at the top and bottom joining on the cup and grip. The object of thickening the plates where the angles and rolled sections occur, is to avoid joining very thin plates to thick ones, which is difficult and causes bad and leaky joints.

Large sheets should be used where possible, as riveting is thereby reduced and there is less chance of leakage, although smaller plates and more rivets tend to stiffen up the shell. The thin sheets are usually cold riveted by 5/16-in. rivets 1⅛-in. pitch, strips of tape soaked in red and white lead being inserted between the sheets at the lap. All rivets above ½-in. diameter have to be driven hot.

The stress on the top sheets is uniform and diminishes in proportion to the rise. The usual rise is one-twentieth of the diameter for small and medium holders and rather less for large ones. The dome being so frail, it requires some support when the pressure of the gas does not assist it in keeping it spherical in shape. This is effected in two ways: (1) by a framing of timber, metal, or a combination of both, in the tank, on which the dome rests when lowered; (2) by a framing of light iron, like an open umbrella, fixed to the dome and top curb with a center post. For small holders the second method is generally employed, as it gives the advantage of the trussed crown, and the disadvantages are reduced to a minimum, but the cost becomes excessive in large holders, and taking all things into consideration a timber frame is recommended for holders more than 70 ft. in diameter.

SPIRAL-GUIDED HOLDERS.—The introduction of spiral guides fixed to the sides of the lifts, such guides taking the place of the vertical standards and cross bracing, is of great interest to gas engineers, the advantage being the saving in the cost of the guide framing, with only a slight increase in the cost of the remaining portions. A great deal of discussion took place as to the stability of holders erected on this principle, and many engineers were of the opinion that they were doomed to fail, but the fact that such a holder has been standing since the system was first introduced, more than twenty years ago, is sufficient proof of stability, at least, for small holders.

More recently larger holders have been constructed and in all cases have given satisfaction. Where two of similar size, one of each type have been erected in the same works and the behavior of each under heavy wind observed, no complaint has been made of the stability of the spiral-guided type. The largest of this type yet designed is now under construction in this country for Buenos Ayres. The steel tank is 200 ft. diameter by 40 ft. 6 ins. deep (one of the largest made) and has a timber framing for resting the crown. There are four lifts, and the total capacity is 4,500,000 cu. ft. The tank consists of seven rows of plates, the bottom row being 1½ ins. thick, riveted by 1½-in. rivets to a bottom angle curb 6 × 6 × 1 in. The guide rail is 4½ ins. high by 4 ins. base. The cost complete would be about £33,500 erected in this country, inclusive of foundations, which works out at £7 9s. per 1,000 cu. ft. complete. Such a holder with outside guide framing would cost at least £2,500 extra on the whole job, or 7½% more.

Perhaps the greatest objection to the spiral-guided holder is its unsightly appearance.

*Portion of paper entitled "Notes on Design and Construction in Gas Works," read Oct. 30, 1911, before the Junior Institution of Engineers, London, Eng.

†London, Eng.

Statistics of the World's Coal Production are given in a bulletin issued by the U. S. Geological Survey, as follows: The figures given are for the year 1910 as far as these were available, including most of the more important producers:

United States (1910)	501,596,378
Great Britain (1910)	296,007,699
Germany (1910)	245,043,120
Austria-Hungary (1909)	54,575,788
France (1910)	42,516,232
Belgium (1910)	26,374,986
Russia and Finland (1910)	24,967,095
Japan (1909	16,505,418
Canada (1910)	12,796,512
China (1909)	13,227,600
India (1909)	13,294,528
New South Wales (1909)	7,862,264
Spain (1909)	4,546,713
Transvaal (1910)	4,446,477
Natal (1910)	2,572,012
New Zealand (1909)	2,140,597
Mexico (1909)	1,432,990
Holland (1909)	1,235,515
Queensland and Victoria	1,119,708
Italy (1909)	611,857
Sweden (1909)	272,056
Cape Colony (1909)	103,519
Tasmania (1909)	93,845
Other countries	5,236,903
Total	1,278,577,813
Percentage of the total produced by the United States	39.2

A Comprehensive Sewage-Treatment Program for the Sanitary District of Chicago.

Physical and legal conditions have combined to impress upon the trustees and engineers of the Sanitary District of Chicago the need for a comprehensive sewage-treatment program looking many years ahead. Such a program has just been made public. It takes the form of a report by Mr. G. M. Wisner, Chief Engineer of the Sanitary District of Chicago, based largely upon studies made at the Thirty-ninth Street Sewage Testing Station by Mr. Langdon Pearse, Assistant Engineer. The report is valuable because it gives important data regarding the present condition of the Drainage Canal and the rivers which receive its waters and points out how the increasing burden on sewage disposal by dilution, under conditions which seem to permit little increase in diluting waters, may be lightened by the construction a few years hence of settling tanks of the Imhoff or Emscher type, supplemented later by sprinkling filters at some of the many settling-tank stations.

Mr. Wisner clearly states that sewage disposal by dilution with Lake Michigan water, the sewage and diluent alike being turned southward to the Mississippi through the Chicago River, the Drainage Canal, the Des Plaines and the Illinois rivers, has been a success, but he makes it equally clear that between the standard of dilution set by the State of Illinois and the limit on diluting water set by the United States the removal of some of the putrescible and oxygen-consuming matter from the sewage will soon be imperative or a nuisance will arise. The apparently speedy exhaustion of the diluting capacity of the water available has been in part due to heavy discharges of industrial wastes. This was not taken into account in the dilution limit, nor was sufficient consideration given to the needs of fish life. The danger of nuisance is increased by the heavy deposits of sludge in portions of the canal, as will appear later in this article. Another interesting point worth noting in this review is that it appears to be more economical to treat a portion of the sewage to (an eventual) relatively high degree than to give all of it partial treatment.

In passing to the details of Mr. Wisner's report it may be mentioned that the report is the outcome of an ordinance passed by the Trustees of the Sanitary District of Chicago in November, 1910, authorizing the appointment of a commission to investigate both the hydro-electric possibilities of the canal and other methods of sewage disposal than dilution, and appropriating $12,000 for salaries and other expenses. On Nov. 25, 1910, Mr. R. R. McCormick, then President of the Trustees, appointed Mr. Wisner, Mr. E. B. Ellicott, Electrical Engineer to the District, and Messrs. W. A. Evans, C. D. Hill and Martin C. Schwab as commissioners. Questions having arisen as to the validity of the appropriation and of the appointments of the three men last named, an investigation of sewage disposal alone has been reported on by Mr. Wisner, the other four merely concurring in the conclusions and recommendations of the report.

In opening his report Mr. Wisner first notes that the population of the District will probably reach 3,000,000 between 1920 and 1922. Since the District Act of 1889 required at least 3.3 cu. ft. per sec. of diluting water for every 1,000 people sewering into the canal, the full 10,000 cu. ft. per sec. of diluting water will be required when 3,000,000 population is reached. The 3.3 dilution was based on studies by Mr. Rudolph Hering, largely on foreign data, and with little regard for fish life or any other consideration but nuisance. That limit "does not provide enough margin for the present enormous load of industrial wastes that are now being discharged into the Chicago River." Nor does it "take account of the effect of deposits of sludge in the river and canal upon the dilution water as it passes over this load of septic sludge."

Dissolved Oxygen.

It appears that even when the dissolved oxygen is insufficient to support fish life nuisance does not inevitably occur; also that to maintain fish life 2½ to 6 parts of dissolved oxygen per million are needed, depending on the character of the fish. During the three summer months of 1911 the canal showed "less than this amount through practically its entire length, and for the lower 10 to 15 miles the oxygen contents averaged from nothing to 10% saturation, which is less than one part per million in the summer time."

The available data seem to show that the condition of the Des Plaines River has markedly improved since the canal was opened in 1900.

Table I. presents the averages of tests made in 1899 and in 1900, just before and just after the canal was opened, and also tests made in July and September, 1911.

Figs. 1, 2 and 3, reproduced from the report, may be studied together as throwing further light on the condition of the [illegible]al and the rivers which receive the sewage [illegible]the district. Fig. 1 shows dissolved oxygen a[illegible]es for June, July and August, 1911, at the [illegible]s indicated. (This map, as also the map con[illegible]ting Fig. 3, will assist the reader in locati[illegible]points mentioned in the report.) Fig. 2 s[illegible] the results of studies less extensive in tim[illegible]t more extensive geographically, since it c[illegible]s the canal and the Chicago, Des Plaines and [illegible]inois rivers. The data here include stability [illegible]non-putrescibility tests, as well as dissolv[illegible]oxygen, besides giving a profile from La[illegible]Michigan to the mouth of the Illinois River. [illegible]3 gives dissolved oxygen determinations of [illegible]ke Michigan water for six months and of canal [illegible]ter at Lockport for eight months in 1911, besides other related data. Tables I. and II. may be consulted in connection with the diagrams. Further refer-

TABLE I.—DISSOLVED OXYGEN IN MAIN CHANNEL AND ILLINOIS RIVER, BEFORE AND AFTER OPENING OF CHANNEL IN 1900.

Sampling Point.	Year.	Dissolved oxygen, parts per million.			Per cent. saturation.		
		1899.	1900.	1911.	1899.	1900.	1911.
Lake Michigan	Aug. 31	..	..	8.9	..	..	105
Main Channel—							
Kedzie Ave.	Aug.	..	2.7	1.4	..	29	15
Controlling works—Above		..	0.6	0.0	..	6	0
" Below		..	6.6	0.3	..	70	3
Power-house	June	..	..	0.0	..	..	0.0
Des Plaines River—							
Joliet, above dam	June	..	2.6	..	..	28	..
" below dam	June	..	7.6	..	..	81	..
Illinois River—							
La Salle	Aug.	8.2	..	3.7	100	..	42
Henry, above dam	Aug.	16.6	..	3.4	210	..	39
Averyville		Aug. 19	Oct.		Aug. 19	Oct.	
		16.1	7.1	5.9	182	75	68
Wesley City		Aug.	Jan.		Aug.	Jan.	
		3.4	10.7	6.2	41	75	71
Pekin	Aug.	4.2	..	6.2	55	..	71
Lancaster Landing	Aug.	9.4	..	..	124	..	..
Copperas Creek Dam	Aug.	9.6	..	4.8	124	..	56
Grafton		Aug.	July	Aug.	Aug.	July	Aug.
		5.9	7.2	6.5	77	94	78
Mississippi River—		July	July	Aug.	July	July	Aug.
Grafton		7.8	7.4	7.5	99	95	90

TABLE II.—RECORD OF DISSOLVED OXYGEN AND PUTRESCIBILITY AT THE POWER-HOUSE OF THE CHICAGO SANITARY DISTRICT, LOCKPORT, 1911.*

1911. Date.	Flow cu. ft. per min. Total.	Per 1,000 pop.	Temperature of water. Fahr.	Cent.	Dissolved oxygen. Parts per million.	Per cent. saturation.	Putrescibility relative stability No.	Remarks.
March 21	320,503	145.7	40	4.5	7.5	58	84	3 ft.
			40	4.5	3.6	28	87	bottom
" 30	323,702	147.1	41	5.0	4.7	37	75	3 ft.
			41	5.0	4.7	37	44	20 ft.
April 6	343,233	156.0	39	4.0	5.4	41	80	3 ft.
			39	4.0	4.1	31	75	35 ft.
" 13	337,477	153.4	47	8.5	4.3	37	75	3 ft.
			48	9.0	4.3	37	75	21 ft.
" 27	354,987	161.1	53	12.0	0.6	6	60	3 ft.
			53	12.0	0.9	8	78	bottom.
May 4	363,164	165.1	52	11.3	2.4	22	71	3 ft.
			52	11.3	1.6	15	10	bottom.
" 10	355,062	161.	58	15.0	0.5	5	32	3 ft.
" 18	402,174	182.8	60	16.0	0.3	4	32	3 ft.
			59	16.0	0.3	4	25	bottom.
" 23	421,311	191.5	60	15.0	0.6	6	80	3 ft.
June 1	401,498	182.5	64	18.0	0.2	2	18	3 ft.
			64	18.3	0.2	2	18	bottom.
" 16	488,364	221.20	65	18.8	0.04	0.4	7	3 ft.
			65	18.8	0.04	0.4	7	bottom.
" 22	435,831	198.1	72	22.4	0.0	0	9	3 ft.
			72	22.8	0.0	0	8	bottom.
" 29	427,467	194.3	69.0	21.0	..	..	9	3 ft.
" 30	420,562	191.2	70.0	21.5	0.1	1	9	3 ft.
July 6	414,517	188.4	73.0	23.0	0.0	0	15	3 ft.
			73.0	23.0	0.0	0	10	bottom.
" 13	445,427	202.5	74.0	23.8	0.0	0	5	3 ft.
			74.0	23.7	0.0	0	5	bottom.
" 19	433,182	196.9	72	22.8	0.0	0	5	3 ft.†
					0.0	0		bottom.
" 19	.....	....	71	22.2	0.0	0	4	3 ft.‡
					0.0	.		bottom.
" 20	430,259	195.6	71	22.1	0.0	0	6	3 ft.§
					0.0	0		bottom.
" 28	443,778	201.7	66	18.9	0.0	0	5	3 ft.
Aug. 3	421,673	191.7	63	17.4	0.2	2	9	3 ft.
			63	17.3	0.2	2	8	bottom.
" 10	323,592	147.1	69	20.9	0.3	3	24	3 ft.
			69	20.8	0.3	3	20	bottom.
" 17	446,059	202.8	73	23.0	0.0	0	6	3 ft.
			73	23.2	0.0	0	4	bottom.
" 24	459,608	208.9	68	20.1	0.0	0	6	3 ft.
			68	20.0	0.0	0	6	bottom.
" 30	431,673	196.2	68	20.0	0.0	0	9	3 ft.
			68	19.9	0.0	0	9	bottom.
Sept. 14	414,546	188.4	68	20.0	0.0	0	9	3 ft.
			67	19.7	0.0	0	9	bottom.
" 21	442,461	201.1	68	20.1	0.0	0	8	3 ft.
			68	20.1	0.0	0	8	bottom.
" 28	414,553	188.4	66	19.0	0.1	1	9	3 ft.
			66	18.9	0.1	1	8	bottom.

*The report also contains a corresponding table of chemical analyses (parts per million) which we summarize as follows (no averages given): Total organic nitrogen, 3.52 on March 30 to 0.65 on Sept. 14. Albuminoid ammonia, 1.09 on April 6 to 0.36 on April 27. Free ammonia, 2.6 on June 1 to 0.46 on June 29; nitrites and nitrates, always absent after June 29, frequently present before that date, the nitrites ranging from 0.6 on April 6 to zero and the nitrates from 0.55 to zero. Total oxygen consumed, 17.1 on July 19 to 2.7 on Aug. 10. Chlorine, 25 on April 6 to 13 on June 1. Total suspended matter, 74 (32 volatile and 42 fixed) on July 19 to 12 (4 volatile and 8 fixed) on Aug. 10. Alkalinity (Ca CO_3) 148 on June 1 to 124 on April 27. Turbidity (silica standard) 120 on May 4 to 35 on Aug. 10, but observations not reported for every date. The cofferdams were removed, opening Western Ave. sewer, on April 15; North Shore Channel in operation May 1.

†9.30 a. m. to 5 p. m., 5 samples.

‡6.30 p. m. to 12.30 a. m., 5 samples.

§2 a. m. to 8 a. m., 5 samples.

...nce to Figs. 1, 2 and 3, and comments on the ...significance of ...e tests, may be found in the following extra... from the report:

From the di...ion standpoint, summer condi...tions are the w...st. At that time the dissolved oxygen in the s...wage is practically exhausted in the larger sew... by the time the water reaches the Chicago Ri...r, and the lake water contains a minimum am...nt of oxygen. Aside from the condition of the sewage, the lake water, varying as it does in temperature from 32 to 74° F. (Chicago Department of Public Works, 1907-9), should also vary in content of dissolved oxygen from 14.7 to 8.6 parts per million, if completely saturated. Observations made at the 39th St. pumping station from May 25 to Aug. 9, 1911, have shown a variation in temperature from 54 to 74° F., and a variation in content of dissolved oxygen from 13.6 to 8.6 parts per million. In other words, in the study of dilution conditions inside the sanitary district, summer conditions will govern since 1,000 cu. ft. per min. during summer weather will not effect any more purification than 445 cu. ft. per min. in the cooler winter months, so far as the content of dissolved oxygen may affect self-purification. The oxygen is the most active agent in purifying the organic matter in the sewage. On the other hand, in the winter, ice may form on the canal and river, and on the Des Plaines and Illinois rivers below, preventing any absorption of oxygen from the air, but this is only for a short period. It is a matter of observation to any one that the water of the canal and the Chicago River is in much better condition during the cold weather than in hot weather.

The effect of the exhaustion of oxygen at Lockport is to keep down the amount of oxygen in the reaches below Lockport and Joliet. There is some aeration from the spillways at the powerhouse. The oxygen supplied by this is diminished by the time Joliet is reached. Below the dam at Jackson St., in Joliet, the water flows swiftly and is much agitated for two or three miles, increasing slightly the amount of dissolved oxygen, which becomes gradually exhausted again by the time the liquid has reached Marseilles. From the result of the two trips (see Fig. 2) it seems probable that sedimentation and oxidation have effected most of the self-purification by the time it has reached Marseilles. This represents the most active period in the cycle of self-purification since in mixtures of sewage and water we have found that the consumption of dissolved oxygen proceeds much faster at the start, and at a diminishing rate, until it is either all exhausted in a very putrescible mixture, or reaches the minimum in a more stable condition. The swift water below the Marseilles dam aerates the liquid very thoroughly, and the oxygen is greedily absorbed. For the best condition of the Illinois River it is desirable to keep as much swift water as is possible.

If a ship canal be created it is probable that, with a more sluggish flow and larger volume of stale sewage, the general condition will not be improved, and sludge deposits will be encouraged.

CANAL.—The analysis of the canal water during the last five months have shown that the present ratio of dilution is approximately correct for the earlier years of the life of this scheme. However, the practical exhaustion of the dissolved oxygen at the powerhouse, and for ten to fifteen miles this side of the powerhouse, indicates that with conditions as they are to-day during the continued hot spells the present ratio is none too great, and that nuisance may begin to develop not only locally, but also in Joliet. However, in Joliet the Illinois and Michigan

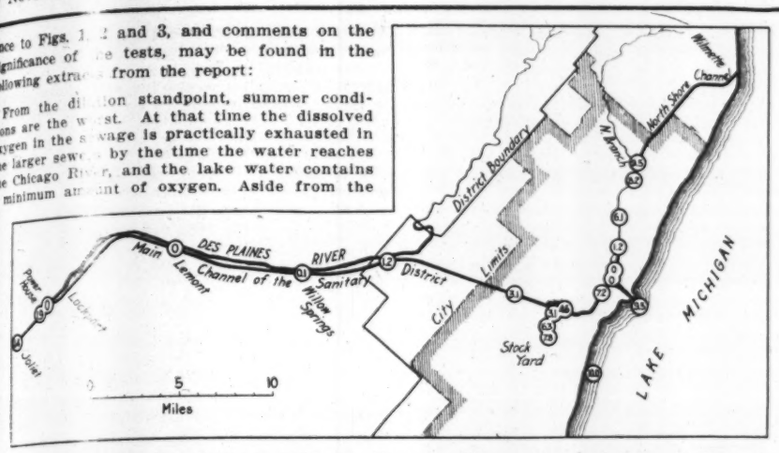
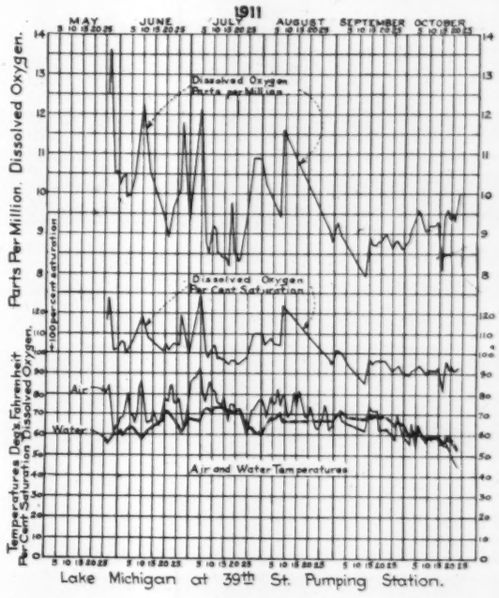


Fig. 1. Average Results of Dissolved Oxygen Tests for Chicago River and Drainage Channel During June, July and August, 1911.

(Flow in cu. ft. per min.; Max., 443,778; Min., 407,118; Av., 423,900.)



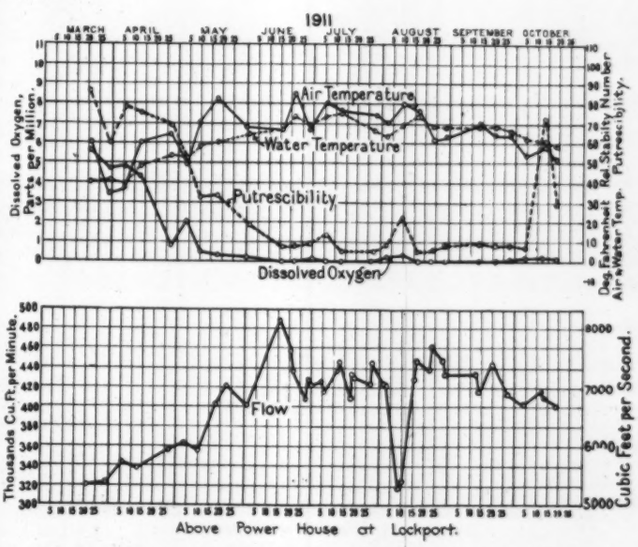


Fig. 3. Observations at 39th St. Pumping Station, Lake Michigan, May to October, 1911, and at the Power House, Lockport, March to October, 1911.

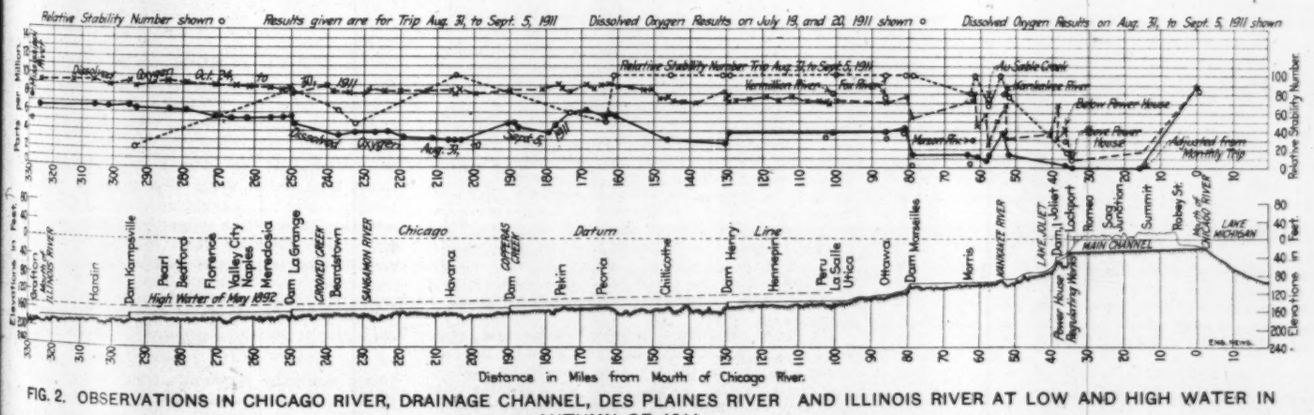

Relative Stability Number shown o
Results given are for Trip Aug. 31, to Sept. 5, 1911
Dissolved Oxygen Results on July 19, and 20, 1911 shown o
Dissolved Oxygen Results on Aug. 31, to Sept. 5, 1911 shown
Relative Stability Number Trip Aug. 31, to Sept. 5, 1911
Vermilion River
Au Sable Creek
Kankakee River
Below Power House
Above Power House
Adjusted from Monthly Trip
Parts per Million
Relative Stability Number
Elevations in Feet
Chicago Datum
High Water of May 1892
MAIN CHANNEL
LAKE MICHIGAN
Distance in Miles from Mouth of Chicago River.

FIG. 2. OBSERVATIONS IN CHICAGO RIVER, DRAINAGE CHANNEL, DES PLAINES RIVER AND ILLINOIS RIVER AT LOW AND HIGH WATER IN AUTUMN OF 1911.

(These observations were made during two trips; the trip extending from Aug. 31 to Sept. 5 was made at low water, when the average reading on the Peoria gage was 8.8; the trip from Oct. 24 to 30 was made at high water, when the average reading of the Peoria gage was 13.9. The gage reads up. Zero of gage is Elev. 435.82, Memphis datum. For the low-water trip, Aug. 31 to Sept. 5, determinations for relative stability as well as for dissolved oxygen were made. In addition to the observations on the two trips already mentioned, some dissolved oxygen tests for July 19 and 20 are indicated by means of small open circles.)

FIGS. 1 TO 3. DISSOLVED OXYGEN IN PARTS PER MILLION IN LAKE MICHIGAN, CHICAGO RIVER, DRAINAGE CHANNEL, DES PLAINES RIVER AND ILLINOIS RIVER AT VARIOUS DATES IN 1911, WITH RELATIVE STABILITY NUMBERS IN SOME INSTANCES AND OTHER RELATED DATA.

Canal is at present an open, stagnant sewer, very much fouler than the Des Plaines River through the town.

The average condition of the canal is shown by Fig. 1, in which is given the average content of dissolved oxygen as determined in June, July and August of this year. It is undoubtedly true that the exhaustion of the dissolved oxygen has been somewhat increased over the normal by the opening of the Western Ave. sewer on April 15, and the operation of the Wilmette pumps since May. By the former action the West Arm of Bubbly Creek was flushed out for the first time to some extent; by the latter the current was increased in the North Branch, bringing down more of the stagnant, septic water from the portion of the river between North Ave. and its junction with the Main Branch. Since March 21, this year, weekly tests of the dissolved oxygen content and putrescibility have been made above and below the power-house at Lockport. These results, together with the flow through the canal, are tabulated in Table II.* They show a highly putrescible mixture from May to September, with the oxygen practically exhausted. The passage through the turbines causes a slight increase in the content of oxygen. Whenever water is passed over the spillway marked aeration ensues. Under average conditions this is sufficient to raise the oxygen content below the power-house to two or more parts per million.

ILLINOIS RIVER.—It is essential for the good condition of the Des Plaines River for many miles below Joliet, and for the Illinois River as well, that the amount of oxygen at Lockport be kept at a maximum, and that as little sludge as possible be allowed to deposit in the canal. In the dry summer weather the oxygen may become very low below Joliet as far as the Marseilles dam, and be practically exhausted there, even with the addition of the flow from the Kankakee.

SIGNIFICANCE OF TESTS.—The most satisfactory tests we have found to be the field tests for dissolved oxygen, supplemented by the putrescibility test. The content of dissolved oxygen gives the present condition of the liquid, a concentration factor so to speak, while the putrescibility test is an indication of the stability of the mixture; that is, it shows the rate at which the dissolved oxygen is undergoing exhaustion. A mixture may be 50% saturated with dissolved oxygen and yet be highly putrescible. This is frequently the case with the samples from the South Branch at Bridgeport (Ashland Ave.). Again, the sample may contain only 30 or 40% of dissolved oxygen, as at Chillicothe, and be comparatively stable. Both factors must be studied side by side, in order to gage the condition of the mixture of sewage and water and the progress of self-purification.

Sludge Deposits in Canal.

CANAL.—At present there is a large deposit of sludge in the forebay of the power-house, extending over two miles north to the controlling works, and into the rock section. This is due to the greater cross-section at that point, and the tendency of the controlling works to act as skimming devices, thus holding back the settling or settled suspended matter. In the rock section there is a little settled matter, slight sludge deposits being found on the sides. In the full-sized section from Willow Springs to Summit, there is a sludge deposit from which rises at times a very noticeable gas ebullition. This deposit is due to the reduced velocity in the large cross-section. The deposit, generally, seems to be a fine silt, containing a small amount of organic matter, similar to the grit that is found in the grit chamber of the 39th St. pumping station.

Fig. 4 shows graphically the deposits in the channel between Western Ave. and the power-house and the velocity in feet per second through the different stages of the canal. The percentage of moisture in these deposits [76.6 at Controlling works on March 21 to 14.5 at Kedzie Ave. on July 28] is much less than that found in the sedimentation tanks at the 39th St. pumping station, where the moisture runs as high as 90%.

The summary of the yardage [of deposits] in the Main Channel is as follows:

	Cu. yds.
Small earth section	398,301
Full earth section	1,347,233
Rock section	0
Forebay	592,800
Total	2,338,334

The specific gravity decreases down stream, and the moisture content increases. The percentage of volatile matter also increases down stream. The content of organic nitrogen also increases down stream, indicating that the fresh organic matter originally deposited has been worked over and unloaded by septic action, and only the more solid mineral and less digestible organic material remains behind.

The effect of the deposits undoubtedly is to reduce the efficiency of the canal by exhausting the dissolved oxygen to oxidize the products of decomposition. In general, the deposits reduce the cross-section, and consequently the available flow of the canal.

For these reasons the sludge from the sewage should be kept out of the canal and river by the introduction of settling tanks as much as is practicable. Of course the silt coming from Lake Michigan during storm periods cannot be avoided, but it can be minimized by dividing the flow as continuously as possible between the Wilmette pumping station, Lawrence Ave. pumping station, the Chicago River and 39th St. pumping station.

General Conditions.

DES PLAINES RIVER ABOVE JOLIET.—During the summer of 1910 in the upper river the pool above the Riverside Dam was a seething septic tank. No flow was passing over the dam, and in the vicinity of the river the odor was complained of. During the summer of 1911, however, conditions were somewhat better. Crappies, or sunfish, were caught at the Ogden Ave. bridge during the past year.

The various communities along the Des Plaines River above Lyons are polluting the river so that complaint arises from Melrose to Lyons from time to time. On the tributary known as Salt Creek, the town of La Grange is building a sprinkling filter to operate on the effluent of an existing septic tank.

DES PLAINES RIVER BELOW JOLIET.—The condition of the Des Plaines River from Joliet down to the junction of the Kankakee must be carefully watched, not only as to nuisance, but also as regards fish life. Owing to the low content of oxygen at times of drought it is very difficult and practically impossible for fish to live there during the summer months.

ILLINOIS AND MICHIGAN CANAL.—The Illinois and Michigan Canal from Chicago to Joliet at present serves no purpose other than to act as a drainage ditch in times of excessive floods, when from the Sag to Joliet it serves a very useful purpose. North of the Sag it is a positive expense, since all sewers emptying into the Main Channel have to pass under the bed of the canal through inverted siphons, which are very costly. Through Joliet the canal is a foul sewer with a sluggish flow of sewage. This canal is to-day a nuisance in Joliet.

ILLINOIS RIVER.—Inquiry along the river shows that in normal summer seasons fish are caught as far up the river as Marseilles, and occasionally farther up the river. The common fish are carp, catfish and bullheads. Less common are bass. Gars can live where other fish cannot, owing to the air bladder with which they are provided. Owing to the great extent of the fish industry in the Illinois River it is essential that the condition of the river, in so far as the Sanitary District is concerned, be kept as good as possible. At the same time, as soon as the Sanitary District begins to provide the treatment for industrial wastes and domestic sewage, the towns in the Des Plaines, Dupage, Fox, Kankakee and Illinois River valleys should impr[illegible] conditions in their localities. The improvem[illegible] of factory wastes should be strictly adher[illegible] for with growth of population and indust[illegible] this great valley it will be impossible for th[illegible]itary Dis-trict to dilute that additional lo[illegible] as well, and fish life will surely suffer. It is [illegible]essary that immediate steps be taken to asc[illegible]n the conditions along the river, the ch[illegible]er of the wastes, extent of sewerage syst[illegible] the content of dissolved oxygen, etc., and tha[illegible]ntinued examination be made year after ye[illegible] by the Sanitary District in order to have the[illegible]ata at hand to refute possible law suits for d[illegible]es to fishing, and the possible reopening of [illegible]e St. Louis case. This is not only a matter of [illegible]itation, but a question of self-defense, protecti[illegible] the root and purpose of the Sanitary District.

POPULATION ON DRAINAGE AREA OF ILLINOIS RIVER.—The total population on the drainage area of the Illinois River and its tributaries in 1910 was 3,907,603, of which 1,566,700 was outside the Sanitary District. Or, put differently, about 60% of this total population, as well as 87% of the sewered population, is within the District. Taking rates of increases in the various sections into account, as well as other factors, the Sanitary District will continue to be the controlling factor in the pollution of the Illinois River, by domestic wastes [evidently by industrial wastes, also.—Ed.]. Nevertheless, the Sanitary District alone cannot keep the river in a satisfactory condition, the report states.

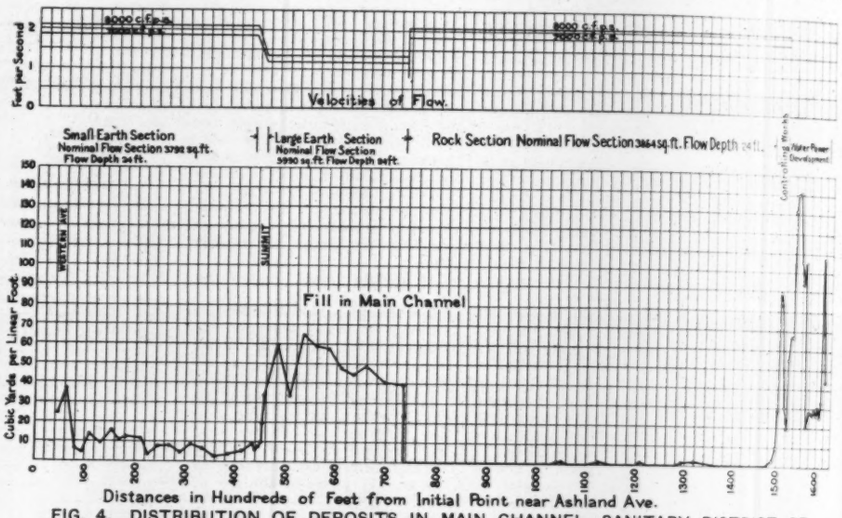

8000 c.f.s.
7000 C.F.S.
Feet per Second
Velocities of Flow.
Small Earth Section
Nominal Flow Section 3798 sq.ft.
Flow Depth 24 ft.
Large Earth Section
Nominal Flow Section
5990 sq. ft. Flow Depth 29ft.
Rock Section Nominal Flow Section 3864 sq.ft. Flow Depth 24ft.
Controlling Works
Water Power Development
WESTERN AVE.
SUMMIT
Fill in Main Channel
Cubic Yards per Linear Foot.
Distances in Hundreds of Feet from Initial Point near Ashland Ave.

FIG. 4. DISTRIBUTION OF DEPOSITS IN MAIN CHANNEL, SANITARY DISTRICT OF CHICAGO, AUGUST, 1911; INCLUDING ALSO VOLUME AND VELOCITY OF FLOW.

SUCCESS OF DILUTION.—The report states emphatically that disposal by dilution has been a success, but that the studies indicate the necessity of action to prevent the overloading of the diluting water.

VOLUME OF SLUDGE.—It is estimated that the Chicago River and Main Drainage Channel are now receiving 137,000 tons annually of dry suspended material from "human sources," of which about 40% may possibly settle. It is also estimated that packing houses and the stock yards contribute about 20% as much suspended matter liable to deposit as comes from the city sewers. In addition, there are large volumes of suspended matters discharged into the North Branch and Main Channel by industrial concerns and from the streets.

Available Methods of Sewage Treatment.

After some further discussion of the conditions in the canal and the rivers, the report next reviews methods of sewage treatment, as regards their availability under the conditions prevailing at Chicago.

Fine screening is dismissed as not being "practicable in large sewer districts." Experiments with scrubbing filters of the mechanical type are considered desirable, to see what can be accomplished in the way of removing suspended matter not removable within practicable limits of sedimentation, these filters to follow tank treatment. The possibilities of reducing putres-

*A footnote to Table II. presents a summary of full sanitary chemical analyses given in the original.—Ed.

cibility by aeration are considered to be such as to make experiments with aeration advisable. "For the present, the only methods of treatment which can be seriously considered are (1) sedimentation for all or considerable portions of the sewage, and (2) sedimentation, sprinkling filters and secondary sedimentation for a smaller portion, as set forth below. We quote again from the report:

Quiescent sedimentation with three hours duration will remove practically all of the settling suspended matter. The reduction in putrescibility is marked, but the indications are that the reduction in putrescibility is not nearly so great as the reduction in suspended matter, the amount varying with the proportion of volatile and fixed matter, as well as the relation of the suspended matter to the total solids and their respective compositions. Tests on various scales indicate from 12 to 100% improvement by settling, with from 30 to 60% removal of the suspended matter. From tests, 15 to 20% improvement may easily be expected. Of the means available to-day for sedimentation, in combination with sludge disposal, the Emscher [Imhoff] type of tank is believed to be the most effective.

Sprinkling filters dosed at a rate to yield from 2,500,000 to 3,000,000 gals. per acre per day (approximately 10,000 people to the acre) will give a very stable effluent containing sufficient oxygen to effect its own self-purification. If this liquid be settled quickly in an Emscher tank the sludge can be readily removed. Secondary settling basins are important in the case of the Sanitary District, with limited dilution, to remove sludge which would otherwise settle behind dams, etc., which tend to hold back the settling suspended matter.

It is important in the case of the Sanitary District to remove the settling suspended matter as far as possible from the sewage before discharging it into the canal or river. The reason is, first, that it is believed that it will materially lessen the amount of water required in the canal per capita to dilute the sewage; second, that it will vastly improve conditions in the canal and rivers below. All the dams along the rivers and canal collect masses of the heavier settling suspended matter, or sludge as it may be called, allowing the skimmed liquid to flow over the crest. Considerable aeration is induced which is probably of some benefit, but with septic conditions above the dam, as is the case at Lockport, Joliet and Marseilles it is believed that such a weir is a detriment unless the sludge can be kept out of the streams. The fermentation of the sludge exhausts the dissolved oxygen and destroys fish life.

COMPARISON OF METHODS.—The duty of the Sanitary District is to keep the canal in at least as good condition as it is at the present time and to improve it as far as is practicable. The two methods available are: partial purification by settling and thorough purification by settling followed by sprinkling filters and secondary settling basins. In order to produce an equal effect on the canal, it is clear that a much greater amount of sewage must be handled, if settled, than if the filters and secondary settling basins are also installed. In other words, about four times the population would have to be handled by settling than by the more complete purification. On the other hand, it is believed that the condition of the canal and Des Plaines River would show a somewhat more marked improvement by the extended use of settling basins than if the lesser amount of sewage were wholly purified. The exact degree of improvement cannot be prophesied. If it is assumed that with the same amount of diluting water available four times as much sewage would be settled as completely purified, the study of cost will determine which is the cheaper method. On the other hand, the sites for filters are decidedly limited, since it is desirable, in the case of a plant of large size in the open, that it be located in a sparsely settled district. Hence, while settling tanks can be built at the end of almost any sewer, the sprinkling filters would be restricted to the regions as yet largely undeveloped. Fig. 5 gives possible future locations for filter sites and settling basins.

The two distinct methods open for extending the capacity of the canal are, first, by settling the sewage of a large population; second, by settling and treating on sprinkling filters the sewage of a much smaller population. By a judicious combination of the two methods it is believed that a most economical development can be secured, when once the allowable flow from Lake Michigan has been definitely settled.

In any case, the installation of settling basins is economical, since sedimentation is a preliminary process necessary for the preparation of the sewage for the sprinkling filter, or other finishing treatment. Settling basin effluents can be intercepted and delivered to the sprinkling filter sites whenever it becomes necessary in the future.

As a basis for comparative estimates, the installation of Emscher tanks for settling the sewage has been selected for the larger sewer districts. The population has been estimated from the ward divisions and precincts, the growth being extended from 1910 to 1930 at the same rate as the growth from 1900 to 1910. The year 1930 has been selected as a suitable year to design to, since whatever goes into service around 1920

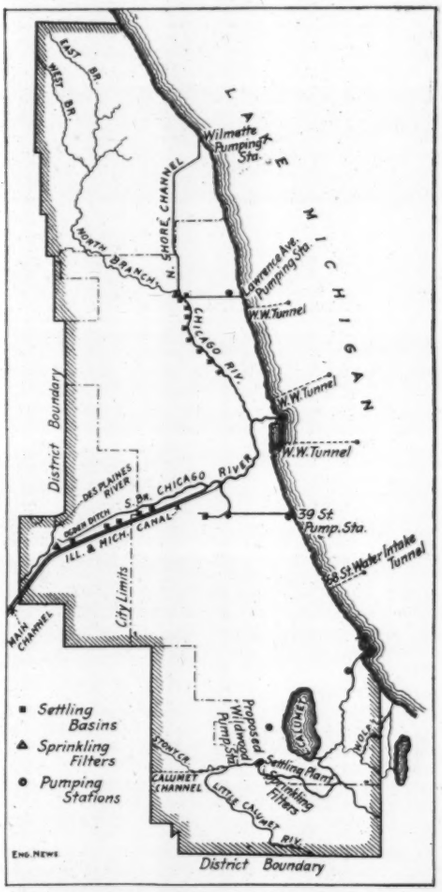


Fig. 5. Sketch Map of Sanitary District of Chicago, Showing Proposed Sites for Settling Basins or Tanks, Sprinkling Filters and Pumping Stations.

(The city water-works intake tunnels and the lake-shore pumping stations for the supply of water or dilute the sewage sent through the Drainage Channel are also shown. The original map showed the location of outfall sewers, up to October, 1911, which are omitted here because of their confusing detail.)

should be capable of serving for ten years thereafter. It will be entirely possible to extend this design to a date beyond 1930, but it is not thought advisable to do so at this time, for the reason that improved methods may be discovered for the artificial purification of sewage that are unknown at this time. For the purpose of these estimates the costs have been made on a per capita basis. The tanks taken as a unit have a nominal settling capacity at a 2-hr. period of about 300 gals. per capita daily and a volume available for sludge storage for a period of six months. This is essentially the same basis as that used at Atlanta, Ga., which agrees very closely with the results obtained in our experimental tank and allows a desirable leeway in flows.

In the design of the tanks it is not recommended that all of the storm water sewage be treated. Practically all of the sewers of Chicago are built on the combined plan. The run-off curve for the 39th St. testing station [not reproduced] illustrates the variation which may occur on a large area. Obviously it would be impossible, from an economical standpoint, to handle all of the sewage up to rates of 46 cu. ft. per sec. per sq. mi. A flow, however, of 20 cu. ft. per sec. per sq. mi. seems a reasonable figure to intercept for treatment. This flow occurred in 1909, only 1.6% of the entire year. The average dry weather flow would seem to be about 6 cu. ft. per sec. per sq. mi., which occurs over 77% of the year.

The matter of the economical point to intercept storm sewage is very debatable. Under the conditions in Chicago, large dry-weather flows may be expected, larger than in most other cities. Three hundred gallons per capita daily seems reasonable as an average figure, which will cover practically the average yearly pumpage at 39th St.

Population and Treatment.

It is estimated that the population of Chicago will reach 3,000,000 about the year 1922, at which time other methods of sewage disposal will have to be installed. The diluting capacity of 10,000 cu. ft. of water will be exhausted somewhat earlier, unless the load of industrial wastes which is now being discharged into the river and canal is taken care of in some other manner. It is, therefore, important to see where possible treatment can be begun. It is estimated that the population will be as follows:

1920	2,900,000
1922	3,000,000
1930	3,500,000
1940	4,100,000
1942	4,200,000

Treatment will have to be begun in 1922, and in 1930 be conducted on the basis equivalent to the complete purification of the sewage of 500,000 people. Assuming purification by sedimentation will effect about 25% improvement, the sewage of 2,000,000 would have to be settled by 1930. If, however, the process were carried further by sprinkling filters, only the sewage of 500,000 people would have to be treated.

TABLE III.—POPULATION FORECASTS AND POPULATIONS WHICH MIGHT BE CARED FOR BY SPRINKLING FILTERS AND BY SETTLING TANKS AT VARIOUS DATES.

		Population cared for by—		
Year.	Total.*	10,000 c.f.p.s. dilution.	Sprinkling filters.	Settling only.
1922	3,000,000	3,000,000	0	0
1930	3,500,000	3,000,000	500,000	2,000,000
1935	3,800,000	3,000,000	800,000	3,200,000
1940	4,100,000	3,000,000	1,100,000	.....
1942	4,200,000	3,000,000	1,200,000	.....

*The population of the Calumet District is estimated at 217,700 in 1922 and 401,180 in 1942.

The indications are that on the basis of settling alone, and the assumption of 25% improvement thereby, settling would not suffice altogether after the year 1935. The deficit, however, would be very small. If the sewage of the entire population in 1942 was settled, this would reduce the equivalent population in dilution to 3,150,000 over that which could be taken care of by dilution. It is not, however, at this time feasible to predict what will occur with the consistent removal of suspended matter. There will, however, be a direct benefit, and settling tanks should be installed and connected with the Calumet-Sag Channel and, as far as circumstances will permit, along the Main Channel and the Chicago River.

SEWER OUTLETS.—A detailed discussion of populations by minor districts is next given. Altogether there are "about 156" municipal sewer outlets in Chicago. For treatment various possible plans are discussed, some of which include the immediate construction of settling basins at many of the sewer outlets. When further treatment becomes desirable an intercepting sewer can be built to gather the effluent from the various tanks and convey it to three possible groups of sprinkling filters.

COST OF PRIMARY SETTLING TANKS OF VARIOUS TYPES.—Estimates of per capita costs of three types of tanks, with four different detention periods, were made. These include the Imhoff or Emscher tank, as designed for Atlanta, Ga.; Dortmund tanks, as used at Gloversville, N. Y., and relatively shallow rectangular tanks, like those at Columbus, O. In each case it is assumed that excavation to the full depth of the tank would be required. These estimates, with comments, are as follows:

Type.	Nominal settling period.	Gals. per capita daily.	Cost per capita.
Emscher	3 hrs.*	200	$1.44
Dortmund	4 hrs.*	200	0.84
Straight flow	3 hrs.	200	0.77
Straight flow	6 hrs.	200	0.58

*In both these periods the sludge storage is not calculated in determining the nominal period of settling.

Although the first cost of the Emscher type of tank is considerably higher than of the others, the amount of sludge removed to be handled is much less, and it is our opinion that this type

of tank is the most suitable and inexpensive in the long run, when the cost of handling the removed sludge is taken into consideration. It also causes less of a local nuisance. For the purpose of this report, therefore, the Emscher tank has been taken as the desired standard in estimating the relative costs for the various-sized plants that are considered:

Plant to handle population	Cost per capita
10,000 to 100,000	$1.50
100,000 to 500,000	1.40

In the larger plants, however, certain allowances will have to be added for a main system of distribution and return conduits, which are not required in the small units, the unit distribution and effluent system being ample.

The land areas allowed for the Imhoff tanks are 0.63-ft. net or 1 ft. gross per head of tributary population. For sludge-drying beds 0.3-ft. net and 0.5 gross area per capita were allowed; also 15 cts. per capita for construction, including appurtenances.

Sludge disposal on land rather than by dumping into the lake is advised. Where local drying and utilization on land near the tanks is impracticable, boats could be used to transport the liquid sludge to available disposal areas. Experiments with sludge incineration are recommended to determine the possibility of reducing its bulk by this means.

As to sprinkling filters and secondary settling tanks we quote from the report as follows:

SPRINKLING FILTERS.—Sprinkling filters are estimated to average 6½ ft. in depth of crushed stone. In general, they can be built for about $28,000 per acre, and will handle 2,500,000 gals. per acre of sewage per day. This is the equivalent of a population of 10,000 people. It, therefore, means that the cost of the sprinkling filters will amount to approximately $2.80 per capita. On large plants this would be somewhat reduced. The incidental cost of appurtenances will be about $1 per capita, making a total of about $3.80 per capita, which is considered reasonable for first-class construction.

SECONDARY SETTLING BASINS.—In connection with sprinkling filters secondary settling basins will be required, and it is recommended that the Emscher tank be used for this purpose also. Although our data are not complete on this, it is estimated that the cost can be taken roughly at 75 cts. per capita. This may be somewhat low, however, as the humus discharged by sprinkling filters varies from month to month, and may run when fresh from 1 to 4 cu. yds. per million gallons when collected in a shallow basin.

The remainder of the report, save for conclusions and recommendations, is devoted to tentative estimates and running comments on costs, governing conditions and needs for the various possible sewage-treatment districts. A few comments on special conditions or needs may be noted.

CALUMET DISTRICT.—An intercepting sewer from the mouth of the Calumet River to a point beyond the beginning of the Calumet-Sag Canal is regarded as an absolute necessity to prevent contamination of the water-supply drawn through the 68th St. crib (see map, Fig. 5). The canal just mentioned will have a capacity of 2,000 cu. ft. per sec., against a maximum river discharge of 16,000 ft. To prevent deposit in this canal and in the main canal, as well as to maintain the dissolved oxygen, settling tanks, followed some years later by sprinkling filters or other treatment, are advised for the sewage from the proposed intercepting sewer.

STOCK YARDS DISTRICT.—Much manure and other organic matter is discharged into the city sewers and into the slips in this district, besides ordinary sewage. Intercepting sewers and central treatment rather than settling tanks for each packing house are recommended. The private basins now in use serve only as grease catchers and the existing screens are too coarse for needed efficiency. There is so much uncertainty as to whether plain settling tanks of the Dortmund type or double-deck Imhoff tanks would be best for use here that a 150,000-gal. experimental plant is advised.

The Chicago Water-Supply.

Rising standards of sanitary science and apparent recent increases in B. coli in that part of the Chicago water-supply drawn from the 68th St. intake indicate the future need of filtration. Emergency treatment of the 68th St. supply should be given so long as present conditions continue. It is estimated that the supply from this source could be treated by hypochlorite of calcium at a cost of 35 cts. per 1,000,000 gals., including 5 lbs. of the chemical to the unit named, as well as interest and depreciation.

Aside from the 68th St. water, the present supply is characterized as "fairly satisfactory," but subject to pollution from shipping and to occasional turbidity. The accompanying estimates of the yearly cost of disinfecting the whole supply from now until 1942 and of filtration from 1922 to 1942 are of interest:

ESTIMATED COST OF DISINFECTION OF ENTIRE WATER-SUPPLY OF CHICAGO.

Year.	Population.	Daily average, 1,000,000 gals.	Annual cost.
1911	2,250,000	450	$57,500
1922	3,000,000	600	76,700
1935	3,800,000	760	97,100
1942	4,200,000	840	107,300
COST OF FILTRATION.			
1922	3,000,000	600	$1,860,000
1935	3,800,000	760	2,280,000
1942	4,200,000	840	2,530,000

Concluding Recommendations.

Other suggestions bearing on the water-supply are included in the final recommendations which close the report and conclude our article. Before giving these we condense a section of the report which will aid in an understanding of the final recommendations.

Dilution at the rate of 10,000 cu. ft. per sec. will be insufficient after 1922, when supplementary works will be needed.

The studies show that "in general it is cheaper to thoroughly purify the sewage of a smaller population than to partially purify the average of a larger population, if the ratio of purification be assumed at 1 to 4." All the projects contemplated need not be started at once nor all of them carried out "to remedy the situation before 1930."

The major capital expenditures up to 1935 and annual expenditures by half decades from 1915 to 1930, allowing 4% for interest and depreciation, are as follows:

	Capital outlay. In interval	Capital outlay. To date.	Annual cost.
1915	$2,018,180	$2,018,180	$80,727
1920	2,449,000	4,467,180	212,587
1925	1,102,000	5,569,180	386,385
1930	2,246,340	7,865,520	710,187
1935	2,065,950	9,931,470	.....

The final recommendations are as follows:

(1) Steps should be taken immediately to develop the scheme outlined herein, and the details of the project which require the earliest attention should be fully developed. The chief engineer should be ordered to prepare plans and specifications for the most immediate projects.

(2) Property and rights of way should be secured at once to cover the probable sites required for the improvements up to 1930.

(3) Wherever the city of Chicago undertakes any new sewer work, the Sanitary District of Chicago should secure provisions for settling plants. In general, such plants should be constructed simultaneously with the sewer outfall.

(4) The discharge of industrial wastes into the river or sewers should be regulated and kept under the control of the Sanitary District. Present conditions should be improved as far as practicable. The removal of the settling suspended matter is absolutely essential. Other treatment may be required where wastes are harmful to fish life or the dilution scheme.

(5) Sewage disposal plants located inside the Sanitary District of Chicago should be subject to the supervision of the district.

(6) The sewers that now discharge into that portion of the West Fork of the South Branch west of the collateral channel should be extended to discharge into the Main Channel, after which the so-called Ogden Ditch should be filled up, as it will serve no useful purpose. If left open this ditch will be an expense, a nuisance, and a breeder of mosquitoes.

(7) Intercepting sewers should be constructed from the mouth of the Calumet River to the beginning of the Calumet-Sag Channel near Wildwood.

(8) Construction of settling basins of the Emscher type should be included in the plan of the Calumet-Sag region. These should be constructed at the outfall of the proposed intercepting sewers, and should be complete before the opening of the Calumet-Sag Channel.

(9) Negotiations should be started with the proper authority in Indiana to induce the municipalities located along the Calumet watershed to take immediate steps to purify the sewage now being discharged into the [illegible]net River and thence into Lake Michigan. [illegible]rk along these lines should be started at on[illegible] order that the project may be completed a[illegible]n as possible.

(10) Intercepting sewers should [illegible]onstructed for the removal of the sewage [illegible]being discharged by Evanston into Lake M[illegible]gan. It is also advisable that a settling basi[illegible] the Emscher type be built at the outlet of [illegible]s intercepting sewer in order to keep the [illegible]tion of the North Shore Channel as good as [illegible]sible. With regard to the towns north of W[illegible]ette, a detailed report should be made cover[illegible] the question of the purification of the [illegible]r and the treatment of the sewage for the va[illegible]us towns as a whole and as units. The cost of [illegible]ation plants for the water-supply, and sewage-di[illegible]posal works, with their annual maintenance, should be compared with the cost of intercepting sewers.

(11) The sanitary conditions of the tributaries of the Illinois River should be investigated, particularly on account of the conditions existing in the industrial towns. The discharge, quality and source of sewage, both industrial and human, should be made a matter of record. Measurements of the flow and chemical analyses of the Illinois River and its tributaries should be systematically kept. Experiments should be made to determine the amount of oxygen required for the various kinds of fish.

(12) The emergency use of hypochlorite of calcium as a sterilizing agent on the water-supply coming from the 68th St. crib should be undertaken at once. With a heavy flood in the Calumet River, accompanied by a southerly wind, the pollution of the water-supply coming from this crib is certain. If possible, the apparatus should be built during the coming winter. This should be built by the city of Chicago, in case the Sanitary District has not the right or power to do so. If this chemical is applied properly it will render the water harmless, but if not applied pending the completion of the Calumet-Sag Channel, the water from this crib will be unsafe at times.

(13) To protect the intakes of the water-supply, the Federal and State authorities have the right to prohibit the discharge of sewage from vessels inside the harbor or its entrance. Regulations should be established and rigidly enforced covering the construction of tanks to carry the refuse, or else the closets should be locked while the vessels are within a minimum distance of, say five miles, from shore. The Government also has a right to establish a dead line around the intakes, with buoys or otherwise, to prohibit all vessels from passing within these buoys, or zones of protection. It is recommended that the Government officials be requested to establish such regulations.

(14) In order to rigidly enforce the provisions of the Federal statute, and the contracts made thereunder for dredging and the disposal of spoil in the lake, action should be taken to prosecute the captains of all tugs who are negligent in carrying such spoil to the distance prescribed by law and contract. While this does not relieve the contractor of his liability, it does impress upon the man in active charge of the work the necessity of obeying the law. The promiscuous dumping of waste materials into the lake, even eight miles from shore, is an economic loss to the community, for if the material were saved by dumping in shoal water behind tight bulkheads it would create values many times in excess of the cost of disposal. This created land would be useful either for park purposes or outer harbor construction. It would also be possible to carry on dredging work and improvements when it would be impossible, on account of storms, to deposit the excavated material eight miles from shore.

(15) The water services in the city of Chicago should be metered.

(16) In connection with the systematic study of the Illinois River, the operation of the 39th St. testing station should be continued, particular study being given to the extension of partial purification by aeration and roughing filters. The laboratory and engineering organization in the Sanitary Department are absolutely essential to the program outlined, both as regards immediate and future developments in construction, operation and maintenance.

(17) It is recommended that no municipalities outside the Sanitary District of Chicago be allowed to discharge into the Drainage Canal, or any of its tributaries, a putrescible sewage; but if municipalities outside the limits of the Sanitary District desire to deliver a non-putrescible effluent from a sewage purification plant to the Drainage Canal, or any of its tributaries, it is recommended that they be encouraged to do so.

Reinforced-Concrete Work At the Railway Terminals, Vera Cruz, Mex.

By R. W. WHITAKER,* Assoc. M. Am. Soc. C. E.

As Mexico has been brought before the public during the past few months on account of the revolutionary trouble, a brief description of a type of construction work that is being carried on in this neighboring republic might be of some interest to engineers.

In April, 1906, a company composed principally of English stockholders was granted a concession for the handling of all imports and exports through the port of Vera Cruz and in order to handle cargo in the most efficient manner it has been necessary to install modern terminal facilities. Construction work was begun in March, 1909, and at the present time nearly all of the structures have been completed. This work involved the expenditure of about twelve millions of dollars (Mexican currency) and comprised the construction of yard tracks, custom warehouses, a union passenger station, engine house, shops and other necessary structures that are essential for a complete terminal.

Although a few of the buildings are of masonry and steel construction, it was deemed advisable to build most of the structures of reinforced concrete, on account of the excessive corrosive action of the atmosphere in this climate. Only those illustrations are shown which the writer thinks will be of interest to engineers.

The roof over the concourse (Fig. 1) is supported by concrete trusses of 12.5 m. (41 ft.) span, spaced 6.5 m. (21.3 ft.) c. to c. Although there are many engineers who decry the use of concrete trusses on account of expensive form work and of the stresses in trusses without diagonal members being more or less indeterminate when subjected to moving loads, it was decided to use the bowstring type of truss without diagonals, on account of the panel loads being practically equal and of their presenting a more pleasing and lighter appearance than the plain slab and beam construction. The trusses have been given a bush hammer finish, while the ceiling beams and panels have been given a cement wash in order to conform to the exterior walls of the station. The skylights were placed in the roof in order to provide ventilation and additional light to the waiting room of the station.

Over the four passenger platforms there are umbrella roofs, which are each 150 m. (493 ft.) long and 7 m. (23 ft.) wide and are supported by columns spaced 5 m. (16.5 ft.) apart. The roofs have been designed to carry dead load only and to resist the wind pressure of the "northera," which occur nearly every week during the winter season. Two of these roofs were unexpectedly subjected to a test loading, during a political demonstration, when several thousand people gathered at the station to witness the arrival of a presidential aspirant. A large number of the more ardent supporters climbed onto the roofs and it was with difficulty that they were prevailed upon to seek a more safe point of vantage.

*Obras del Puerto No. 2, Vera Cruz, Mexico.

FIG. 1. REINFORCED-CONCRETE ROOF OVER CONCOURSE SUPPORTED BY BOW STRING TRUSSES.

FIG. 2. REINFORCED-CONCRETE UMBRELLA SHEDS, VERA CRUZ, MEX.

The bridge over the terminal property is 303 m. (993 ft.) in length and is for foot traffic only. The railing shown in the illustration was adopted on account of being a very economical type and of harmonizing with the main part of the structure. The top and bottom rails, together with the uprights for one panel, were molded on the ground and were afterwards set in the forms for the railing posts, which were poured in place. The panels are 2 m. (6.5 ft.) in length and are light enough in weight so that three men could handle them without being obliged to use hoists of any kind.

Although the Mexican labor employed on this work had no previous experience in reinforced-concrete construction, the results obtained have been very gratifying.

Chemical Precipitation as a Means of Sewage Treatment is still being adopted for new installations in England. It is to form an important part of a new sewage purification plant to deal with a portion of the sewage of the urban district of Surbiton, where the sewage will pass, in succession, through detritus tanks, chemical precipitation tanks, primary and secondary filters (evidently percolating), and humus or final settling tanks. Storm water tanks will also be provided, to afford sedimentation for a portion of the excess flow in times of rainfall. The following outline of the new works is from the "Engineering Supplement" of the London "Times" for Oct. 19, 1911 (Imp. changed to U. S. gals.):

The detritus tanks will be in duplicate, and will each have a capacity of 14,400 gals. The detritus will be removed by a grab worked by hand. There will be six precipitation tanks designed to provide for a future population of 30,000. The total capacity is 648,000 gals., or slightly in excess of 12 hours' flow. The precipitant proposed to be used is aluminoferric. Provision is to be made for draining the sludge to a sludge sump, and for siphoning off by floating arms the top half of the supernatant water to an empty tank, the remainder going to the pump well. The storm-water tanks, of which there will be two, will have a capacity of 216,000 gals., or 18,000 gals. in excess of one-quarter of the present dry-weather flow.

There will be five primary filters, 81 ft. in diameter and 4 ft. deep, with a total capacity of 3,870 cu. yds., while the secondary filters will be exactly similar in all respects, except that they will be filled wih slightly smaller medium. The latter can also be utilized as primary filters if so desired by means of a special automatic diverting apparatus. The primary and secondary filters are designed to treat 1,260,000 gals. at the rate of 162 gals. per cu. yd. per day, and the filling media will be clinker between 1-in. and ½-in. gage.

The humus tanks are designed to deal with 108,000 gals., or two hours' flow. The removal of the deposit will be effected by a chain sludge pump. The sludge will be raised from the sludge sump by compressed air to a tank where the heavier portion will settle and the top liquid will be drained off to the main pump well. The sludge will then gravitate into rams, and will be forced by compressed air into the sludge presses. There will be four presses of a capacity of 4.4 tons [9,856 lbs.] of pressed sludge cake per charge. After treatment the effluent will be conveyed direct to the River Thames by a 30-in. effluent pipe to discharge a maximum of 9,900,000 gals. a day.

Mr. Henry T. Mather is the engineer to the council, and Mr. Stanley Wilkinson is the resident engineer.

FIG. 3. LONG REINFORCED-CONCRETE FOOTBRIDGE APPROACH TO STATION, VERA CRUZ, MEX.

Causes of Rail Corrugation.

By CHARLES K. MOHLER.*

Rail wear in the form of waves or corrugations on the head is an aggravating and persistent maintenance trouble very frequently met with under certain operating conditions of steam, and in electric transportation. It often develops very rapidly with electric traction, and the rail may become in such condition under heavy traffic that its renewal is made necessary after a couple of years' service.

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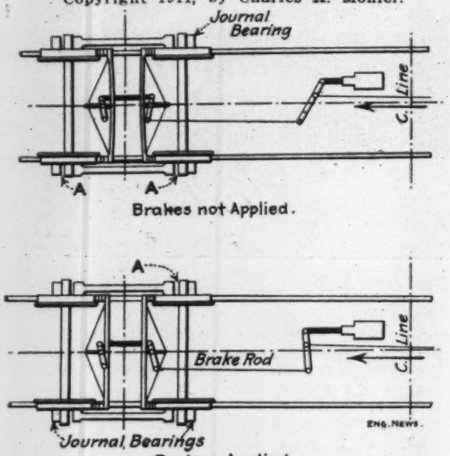

Fig. 1. Plan Showing Relative Position of Axles and Bearings of a Steam Car (In Motion) with Brakes Released and Applied.

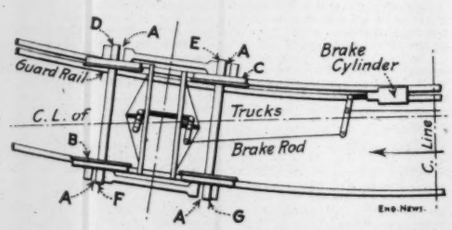

Fig. 2. Plan Showing Relative Position of Axles and Bearings of a Steam Car or Trail Car on a Curve. (Brakes Not Applied.)

Fig. 3. Corrugation of Inner Rail on a Light Curve of an Elevated Railway (Chicago). (Train movement from right to left.)

Many theories have been advanced to account for rail corrugation, but there does not seem to be any remedy applied, based on whatever assumption, that gives effective relief. It is questionable if the theories so far advanced account fully for the true cause of corrugation. A report presented to the International Street and Interurban Railway Association (1910) enumerated eight factors as being the principal causes of rail corrugation as follows: "(1) the quality of the rail (recent observations indicate that this is the principal cause of corrugation); (2) the use of hard tires; (3) too rapid braking at high speeds; (4) too rapid acceleration; (5) too high speed; (6) sharp curves; (7) character of track substructure; (8) nosing or side oscillation of the car. This last can be produced in various ways: (a) rigid mounting of two wheels of unequal diameter on the same pair of axles; (b) unequal diameter of the two wheels on the same axle; (c) difference in elevation of the two rails; (d) unequal division of the weight; (e) variations in resilience of the different springs of the same car; (f) play in the journal boxes; (g) irregularity in the gage of the wheels; (h) irregularities in the surface of the track; (i) effect of torsion due to the method of motor support; (j) axles out of square; (k) bad construction of the car; (l) too short wheel base."

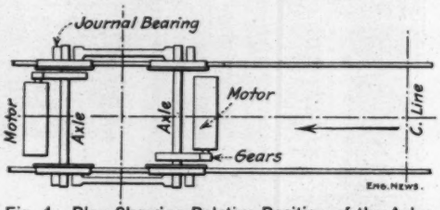

Fig. 4. Plan Showing Relative Position of the Axles and Bearings of an Electrically Operated Car Truck on Straight Track.

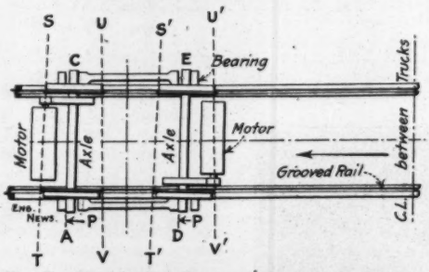

Fig. 5. Electric Car Truck on Track with Grooved Rail Head.

To the above list may be added the "vibration" theory, and some others of like character which have been advanced at different times. All of the above may be dismissed from consideration as the true cause of rail corrugation. Many of them have a secondary influence. As proof that they are not primary causes, we need only consider the fact that all (with the exception of torsion) are present and should be just as potent in the production of corrugation under one form of traction and operation as in another. Under certain conditions corrugation is confined almost wholly to electric traction. In other cases it will develop alike with steam and electric traction but not from any of the above causes alone. Sometimes it will develop on one curve and not on another. It will develop on straight track under certain causes whether with electric or steam propulsion. On straight track corrugation will be found occurring first on one rail and then on the other. It may be asked why the development is so marked in electric traction? Though corrugation is not wholly absent in steam traction, when it does occur the cause is often different from that producing most of it under electric traction. By electric operation is meant that in which all or a majority of the wheels are drivers. Where electric locomotives are used, or where only a few of the total number of wheels are drivers, the operation would be practically the same as steam.

FIG. 6. CORRUGATIONS OF GROOVED RAIL ON STRAIGHT SURFACE TRACKS (CHICAGO).
(On Wabash Ave. south of Adams St., west rail of west track; car movement from left to right.)

Steam Operation.

STRAIGHT TRACK—With steam traction, typical rail corrugation may be produced on straight track through the application of the brakes. Fig. 1 is an outline sketch of the running gear and brake rigging of a freight car. With brakes released and the car in motion the rear face of the axles will be in contact with the journal bearings. The play between the axle and bearings in each case (as at A) is shown very much exaggerated by the open space in front of the axle. The direction of movement is shown by the arrows. The lower plan

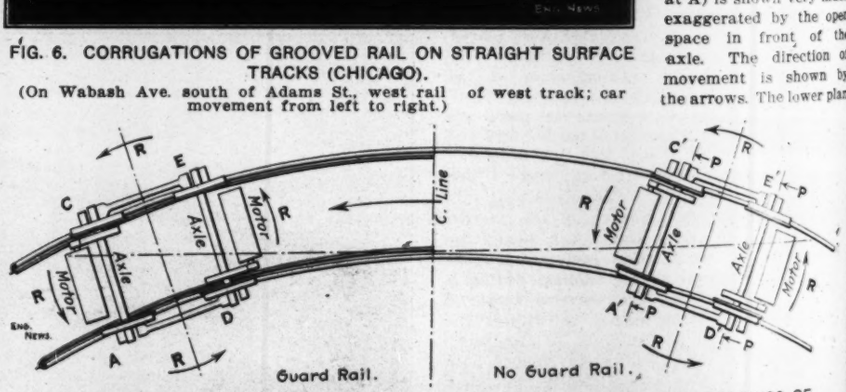

FIG. 7. PLAN SHOWING POSITION OF AXLES, WHEELS AND JOURNAL BEARINGS OF ELECTRICALLY OPERATED TRUCKS ON CURVED TRACK WITH AND WITHOUT GUARD RAILS.

shows the brakes applied and the leading axle thrust forward by the brake pressure so that the front faces of the rear axle brought in contact with the journal bearings. In the rigging shown and also that for passenger cars and trailers for elevated railway service the pull of the brake rods from the cylinder levers is not through the centers or pivots of the trucks but about a foot to one side. The brake rod pull as a result has a tendency to rotate the truck around the pivot and produce flange pressures and throw the axle and wheels diagonal to the track which will result in side slipping of the wheels on the rails. Owing to the flange pressure and possible coning of the wheels, longitudinal slipping will usually result as well as transverse.

In normal operating condition, the play between the axle and the journal, and between the journal and the frame, is about ¼ in. on passenger trail cars of elevated railways. The end play of the axle is about ⅝-in. Play between the wheel gages and the track gage is from ⅛ to 1 in. or more. The eccentric or side pull of the brake lever rods will have a tendency to rotate both trucks in the same direction, and produce a flange pressure of about 450 lbs. When the truck is thus swung around and all of the play and lost motion taken up it is possible for the front axle, with inside hung brakes (or the rear with outside hung brakes), to take a position about 1⅝-in. out of normal to the rail. That would require the wheels to slip about ¼-in. transversely to every foot they rolled forward. On account of the elasticity of the trucks and tracks, the slipping will rarely be continuous but will usually take place at intervals. A joint, a hard or soft spot, or an angle in the track would have a tendency to produce or retard slipping, as the case may be, at that particular point. When once started it may be repeated and extended in corrugations where the brakes are always applied, such as station stops and down grades.

FIG. 8. LONG CORRUGATIONS OF GROOVED RAIL ON CURVED SURFACE TRACK (CHICAGO).

(This is at the beginning of a curve, and the length of corrugation decreases as the radius decreases. The car movement is from right to left.)

CURVED TRACK (WITHOUT GUARD RAIL).—On curved track, with a pair of wheels rigidly fastened to the same axle one or the other of the wheels must slip along the length of the rail unless the outer wheel is coned the exact amount to roll on that particular curve [which it never is.—Ed.] Fig. 2 (guard rail disregarded) shows the normal relation of the axle to the journals when the brakes are not applied and the movement is in the direction of the arrow. The clearance between axle and bearing is indicated at (A). The position shown is brought about as follows: When the truck reaches curved track the flange of the outer leading wheel (B) is brought in contact with the side of the rail, the direction is changed and the rear inside wheel (C) becomes practically the center of rotation; the normal tendency is for the wheels to roll straight ahead with the rear face of the axle in contact with the journal bearings, as shown in Fig. 1, brakes not applied. When the truck begins to rotate and change direction, the pressure and contact on the rear face of the inner journals will be gradually changed until they are transferred to the front of the axle as shown at (D) and (E); after that the continued rotation of the truck will produce a change in the direction of the axles by holding or pulling (D) and (E) back to make up the difference in distance which has to be traveled on the inner and outer rail around the curve.

Conditions may be such that (F) or (B) [(F) and (B) both refer to same wheel] and (G) are pushed to slip forward. With the axles moving at an angle with the radius, transverse slipping of the wheels on the rails takes place at the same time. Without guard rails the truck is guided around with the flange of the outer leading wheel in close contact with the rail. Under those conditions the slipping is comparatively uniform. Most of the irregular transverse, as well as longitudinal slipping is on the inner rail. That is usually indicated by the rapid surface wear and cold rolling. The rear axle is usually more nearly radial to the curve than the front.

The brake-rod pull increases the flange pressure on a curve to the right and reduces it on a curve to the left with the rigging shown. A condition which should tend to favor the production of corrugation is when the brake pressure is such as to carry forward the leading axle of the truck with an inside hung brake (or the rear axle with outside hung brake), and maintain it in such position that a little increase or decrease of the rolling resistance or the brake pressure will throw the axle ahead or back in the journal bearings.

CURVED TRACK (WITH GUARD RAILS).—Probably among the most pronounced factors in causing excessive rail wear and corrugation are (1) the inner guard rails on curves, and (2) the grooved rail on both curved and straight track. Fig. 2 shows curved track with an inner guard rail. The guard rail forms a groove and the inner leading wheel becomes the guide to change the direction of the truck, while (C) is the approximate center of rotation. In addition to the flange pressure required to slip the wheels longitudinally and rotate the truck to follow the curve, there is the lateral pressure produced by the wheels on the leading axle rolling at an angle to the longitudinal axis of the rails. This flange friction on the diameter of the wheel larger than the tread will have a tendency to carry the inner wheel ahead at a faster rate than wheel (B). The former traveling on the inner rail already travels too fast for (B). That of course is only an added factor to interfere with the normal movement. The truck frame and track must be considerably stressed before the inner wheel will be forced to slip back or the outer wheel (B) slip forward on the rail.

As above indicated, most of the slipping may be expected to take place on the inner rail. With the inner guard this cannot well occur until the flange pressure is largely reduced. Transverse slipping must occur at the inner wheel. When transverse slipping takes place and the flange pressure is reduced, the stress set up in the truck frame, track, etc., will probably cause this wheel to slide longitudinally or "kick" back and relieve the stresses for the instant. As the forward motion continues, the track and truck again become stressed and the adjustment is repeated. Any irregularities in the surface and alinement of the track, variation in gage of the track and distance of the guard from the running rail will serve to hasten or retard adjustment at the different points. That is probably why corrugation when once started usually extends in a fairly regular wave length and rapidly increases.

Fig. 3 shows some bad rail corrugation on the Northwestern Elevated R. R. (Chicago) that has developed under the guard rail conditions just described. From 70 to 75% of the wheels passing over this track are trailer wheels (without power applied). Fig. 9 shows how the excessive top wear of the rail terminates with the inside guard rail. This case is a short distance beyond the termination of curvature on an upgrade. The excess wear is shown by the flat head. The termination of the guard rail contact is shown by the abrupt change to the original rounded surface with top wear almost absent. The actual excess wear opposite the guard rail contact in this case was about ¼ in. The termination of excess rail wear coincident with the termination of the inner guard rail contact is a condition found to be universal on all elevated roads in Chicago.

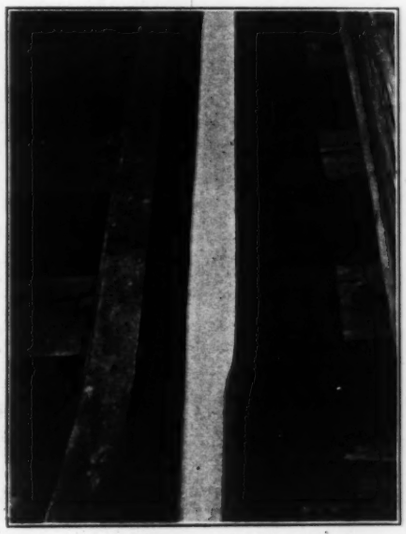

Fig. 9. Excess Wear on Top of Rail Opposite Guard Rail Contact.

The abrupt termination with the end of guard-rail contact is most marked on the leaving end of the curve. (Direction of movement from top towards the bottom.)

Electrical Operation.

STRAIGHT TRACK.—Fig. 4 is a sketch of an electrically propelled truck. One essential difference between such a truck and one on a steam railway car (or a trail car) is the relation of the contacts and lost motion play between the axle and journal bearing. In the electrically driven truck the axle moves forward and its front face is in contact with the bearing, while on the trail or steam hauled car the rear face of the axles are in contact with the bearings. (Figs. 4 and 1, brakes not applied.) With straight track of T or flat-lip girder rail, on uniform solid foundation and smooth clean rolling surface, corrugation should be slow in developing.

With a groove lip or guard rail, as shown in Fig. 5, straight track corrugation may develop in a short time. When for any reason the flange of the wheel is brought in close contact with the groove lip or guard of the rail there is a tendency for that wheel to rotate on the larger diameter of the flange. Let (A) and (D) be the wheels, the flanges of which have come in contact with the lip of the groove. The approxi-

mate coning or difference in rolling diameter is shown by T S. U V) and (T' S.' U' V'.) Then (A) and (D) will have a tendency to roll faster than (C) and (E) and take a diagonal position on the track. This is easily accomplished on account of the lost motion and play between the axle and journal and frame. All of the tractive force could and probably would be transmitted through the journals (A) and (D) which would have a tendency to swivel the truck and increase the diagonal direction which the axles can take by the amount of the journal end play and the track gage play.

As soon as the wheels roll at an angle to the longitudinal axis of the rail, transverse slipping on the rails takes place. Under the tendency to roll on the larger diameter of the flange of the leading wheels (A) and (D) they may gain enough in rotation to cause the lagging wheels (C) and (E) rolling on the smaller diameter to slip longitudinally at the same time they slip transversely. The longitudinal added to the transverse slipping would produce more rapid wear. Longitudinal

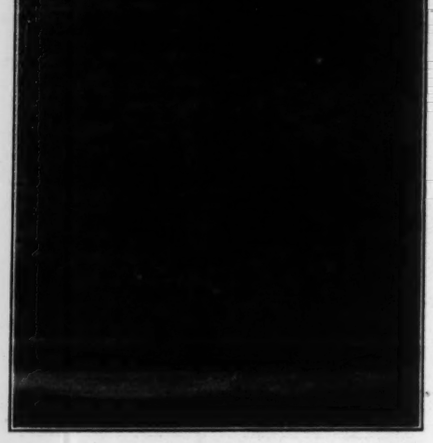

Fig. 10. Notch Worn in Near Rail as a Result of the Wheel Riding on its Flange Through a Shallow Groove in the Opposite Rail.
(Movement from left to right.)

slipping will wrench the trucks and axles still more out of their normal position, causing greater transverse slipping and wear. It will also help to prevent the lagging wheels from regaining their normal position.

Fig 6 shows corrugations on straight track on the surface line on Wabash Ave. (Chicago), south of Adams St.; the direction of movement is from left to right. This track was laid during the summer of 1908 and photographed July 16, 1911. It carries some of the heaviest traffic in the city. Corrugation is developing quite extensively throughout the line. Probably one of the prime factors in the production of rail corrugation on straight track under electric traction is the inherent tendency under disturbing influences for the axles and driving wheels to take a position diagonal to the rail, resulting in transverse slipping. The cases of corrugation on straight surface lines observed in Chicago show close contact and flange wear either on the lip or guard of the opposite rail, or on the gage side of the rail on which the corrugation occurs.

Fig. 10 is a photograph taken at Madison and La Salle Sts. It shows a concrete example of the influence of rail wear when the rolling diameter of the opposite wheel is increased by the flange rolling in the bottom of a shallow grove. The bright polished spot on the near rail shows where excess wear is taking place. As shown by sighting along the line of paving blocks it begins some distance before the frog point is reached, almost opposite the beginning of the shallow grove. The car movement is from left to right. Worn notches of the kind shown are very common. The depth of this particular notch, as the result of 1 year and 10 months wear under the heavy traffic, is about 0.07 in.

While the lip of a grooved rail and the guard rail on T-rail track are unquestionably very active factors in the production of corrugation they are not wholly responsible, as it often develops where there is no such flange contact. Soft spots in the track, low joints, uneven and bad surface and variations in gage may be responsible for starting corrugation under the tendency of the propelling axle and wheels to take and maintain a diagonal position to the track. The vibration of the motor gear is probably instrumental in causing initial slipping at critical points. The accumulation of grit and mud on the rail is still another aggravating factor. After corrugation is actually started on one rail it is gradually extended and finally begins to develop on the rail opposite.

The tendency of the truck to oscillate or "nose" from side to side, thereby throwing the rotation of the wheels out of the parallel longitudinal axis of the rail, combined with the other causes above enumerated, is another contributing factor. Initial corrugation will develop on one side of the track and then on the other within a short distance.

CURVED TRACK (WITHOUT GUARD RAIL). Fig. 7 shows motor-driven trucks on a curve, with the direction of movement as shown by the long arrow. The right half shows the track without and the left half with the guard rail. When power is applied to the axles by means of the spur gear commonly employed, the position taken and the reactions relative to the journals are as shown in Fig. 7. Generally speaking, the tractive power will be transmitted through the inner journals (A') and (D') unless (C') and (E') are rolling on a diameter enough larger than (A') and (D') to make up the greater distance they have to travel around the outside of the curve. It is impossible to have the coning on the outside wheels fit all curves of different lengths and radius. The unusual conditions which obtain in electric traction under the case being considered are as follows: (1) in following the curve the truck must rotate in the direction of the arrows (R). The tractive power being applied at (A') and (D') tends to rotate the trucks in the opposite direction; (2) to the flange reaction caused by the resistance of the wheels to slipping on the rails as the truck has to change direction, must be added that caused by the propelling force all being applied to one side of the truck through the wheels (A') and (D').

The leading axle will have a tendency to take an extreme diagonal position resulting in a large amount of transverse slipping in addition to the longitudinal, giving still greater flange reaction. The position which the rear axle will take on account of the journal play may bring it in almost a radial position. When in its normal position the outer end (E') is ahead, and the inner end (D') behind the radial line passing through the center of the axle. On (A'C') the journal play tends to allow the increase of the already advanced position of the inner end (A') and displace the outer end (C') still farther back of the radial line. On track without a guard rail and of relatively light curvature, corrugation may not develop at all, on account of the outside leading wheel of the truck guiding the truck and tending to roll on the larger diameter of the coning or flange. That will usually result in the inner wheel slipping or "kicking" back when longitudinal slipping has to take place.

CURVED TRACK (WITH GUARD RAIL).—The general conditions of operation with the guard rail are the same as for those just described, with the important exception that the flange of the inside leading wheel (A) guides the truck and changes its direction instead of the leading outside wheel (C). This is shown at the left of Fig. 7. The outside leading wheel flange no longer comes in contact with the outer rail to increase the rolling diameter and increase the friction to carry the truck forward and rotate it in the direction of the arrow (R). The flange friction and pressure are transferred to the inner wheel (A). The outer wheel (C) is largely a floater or idler. The axles must change their direction changed as they pass the curve. That cannot take place unless (C) and (E) are pushed ahead or (A) and (D) pushed back. But (C) and (E) cannot be pushed ahead until the truck has been rotated and journals of (C) and (E) advanced sufficiently to take up the play or lost motion and produce rear contact pressure greater than the resistance to slipping ahead. As soon, however, as contact is established and pressure exerted on the rear face of the axles at (C) and (E) there will be the tendency to produce added tractive acceleration and rotative force to carry (A) and (D) ahead; (A) cannot readily slip or kick back on account of the extra flange friction against the guard rail.

All of the actual propelling force must of necessity pass through (A) and (D), as the rotative change in direction of the truck always leaves the outer wheels (C) and (E) lagging behind. By the time enough pressure has accumulated on the rear of the outer ends of the axles (C) and (E) to force slipping or "kicking" back of the inner wheels, the truck frame, track, etc., will be under heavy stress. When slipping starts on account of sliding friction being less than rolling friction the spring and elasticity of the stressed truck and track will carry the wheels to a new position largely relieving the stresses, only to be repeated in a new cycle of more or less regularity. The great amount of stress brought on the track and fastenings through the influence of the guard rail is well shown in Fig. 9. The bolts of these guard rails are continually working loose and are frequently broken. When a wheel passes over the corrugation wave it probably rolls over the ridge with most of the slipping taking place in the depression.

It is believed that the development of corrugation and excessive wear at curves with guard rails is chargeable largely, if not wholly, to the inner guard rail contact with the inner wheel of the leading axle. Excessive inner rail wear persistently follows the inner guard rail. On short radius curves the adjustments have to take place so continuously and at such short intervals that the wear is quite uniform instead of becoming concentrated in spots. That is well illustrated in Fig. 8, which shows the corrugation at the beginning of the curve on Garland Court and Randolph St., Chicago. At the entrance to the transition the corrugations have long intervals. As the length of the radius decreases, the length of the corrugations decrease until the body of the curve is reached, and there they disappear. Corrugations of shorter length have developed on the outer rail of the curve well beyond where they first appear on the inner. The movement is from right to left. The depth of the long corrugations on the near or inside rail are about 0.09 in. The rail has been in service about 2 years and 8 months under very heavy traffic.

While more might be said in detail, and minor factors enlarged upon, it is believed enough has been said to point out the main and important factors in the production of excessive rail wear and corrugation. Further investigation and study may develop that some factors have been overlooked and others misjudged. It is hoped, however, that the lines indicated will prove a fruitful field for observation and study.

The Greatest Concentrated Rainfall on Record is reported by the Catholic meterological station at Manila, P. I., as having occurred in Western Luzon last July and August. There were three typhoons in that district between July 11 and Aug. 2, 1911, the heaviest of which raged from July 11 to 17. During the first three days of that period the total rainfall recorded at Baguio was 88 ins., 32 ins. of which is reported to have fallen during the first 24 hrs. Baguio is the mountain resort in the province of Benguet, at an elevation of several thousand feet. The road leading to it, which was built a few years ago at large expense and difficulty, is said to have suffered great injury from the floods following the storm. In our issue of Nov. 18, 1909, p. 551, we noted a rainfall in Jamaica, W. I., of 48 ins. in four days, which was then thought to be the heaviest rainfall ever recorded for a limited period.

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The Longest and Lightest Reinforced-Concrete Arch.

In the European technical press there may be continually found implications that the American structural engineer is over bold in his attempts to get just a little more out of every structure than its material or the theory of stress naturally permits. In so far as it concerns structural steel practice—which has developed, in the size of structure at least, far beyond European standards—this criticism may be valid, but in that newer structural material, reinforced concrete, the shoe is decidedly on the other foot. Every week there come to us from the other side of the ocean descriptions of reinforced-concrete buildings and bridges which would not be allowed under any law or specification in this country and which, were they allowed, we feel sure few American engineers would care to be responsible for.

A very striking example of such a design is given in the description, on another page of this issue of the new Ponte del Risorgimento, the longest reinforced-concrete arch span in the world, recently completed across the historic Tiber at Rome.

While no record-breaking structure was ever designed by merely enlarging the details of some successful smaller structure, the very peculiarities in the behavior of large masses of material would seem to require a respectful regard for precedent. Such a regard the designers of the Tiber bridge have considered superfluous. The 328-ft. arch which they have built of reinforced-concrete resembles in design no other structure of even one-third its size, and furthermore goes counter to at least one of the hitherto accepted fundamentals of arch design. Unfortunately the account which we have been able to prepare leaves vague many of the details of design and construction, but the information which is available is sufficient to show the novelty of the structure.

One of the first, if not the first requirement for an arch bridge is a stable foundation. Unless the arch rib can be held firm against any lateral or vertical movement the stresses are indeterminate and failure is to be expected. In smaller bridges, transmitting moderate loads to the abutments, the resisting power of a piled earth or even of an unpiled dry gravel or sand is often considered sufficient, but with the enormous loads transmitted through long arches, nothing but solid rock foundations to receive the arch thrust would ordinarily be considered safe. In the Tiber bridge this principle has been entirely disregarded and a species of floating foundation, a combination of a slab and buttresses resting on piles, has been superposed on a soil so insecure that sheeting had to be provided to keep it from being washed out by the river.

Because of this very doubtful footing, loads carried to it had to be as low as possible, which fact, together with the lack of funds and a need for haste in the completion of the bridge, explains the very light nature of the arch itself. Virtually the bridge is a closed cellular box of reinforced concrete, with an arched plate for a bottom face. Just how much of the stress is assumed to be carried by the vertical ribs extending upward from the arch rib proper and how much by that rib is not stated in our information, nor is it apparent on what grounds such an assumption might be based. Here, in fact, is the doubtful point in the design, which we hope will be cleared up in some later announcements from the designing engineers.

In any event it is interesting to compare this rib with that of the next largest concrete arch, the 320-ft. span of the Grafton Bridge at Auckland, New Zealand. This latter arch is about half the width as that at Rome, some 8 ft. less in span, 58 ft. greater in rise and is designed to carry about the same loads. It consists of two three-hinged reinforced-concrete arch ribs, spaced 24 ft. c. to c., 4 ft. 3 ins. wide, and varying in depth from 6 ft. at the abutment through 10 ft. at haunches to 5 ft. 6 in. at the crown. The roadway is supported on columns which do not aid the ribs in carrying load. Compare with this the 328-ft. Risorgimento arch, with a continuous rib over its whole 66 ft. width, 8 and 11 ins. deep at the crown and 20 ins. at the abutment, indeterminately assisted by seven 8-in. vertical longitudinal walls, extending from the rib to the floor slabs. Very evidently either the New Zealand authorities were over careful or the Italian engineers assumed a large proportion of the load to be arched through the longitudinal walls to the abutments.

However suspicious we may be of this design, there is the unanswerable argument that the bridge is standing and carrying its intended loading, and this is true of most of the reinforced-concrete work in Europe. As long as their structures stand, these continental reinforced concrete specialists have somewhat the better of any argument concerning their daring efforts. The best the more conservative American can do is to admire the nerve that can produce such designs and to study the construction methods that make them possible.

A Paper on the Design of Gas-Holder Frames.

The paper on the design of gas-holder guide frames which we publish in this issue, is a notable contribution in that it covers a field of structural design which has been hardly touched hitherto in engineering literature. It is one of the failings of the technical literature of to-day that it goes on recording over and over again work done in fields of engineering which have become thoroughly familiar to the profession, while it leaves many other fields almost unworked.

The art of gas-holder design has been developed by engineers who have worked alone, without publicity, and with almost no published records of their methods. The author, and the firm with which he is connected, Messrs. R. D. Wood & Co., deserve credit for their breadth of view and professional spirit, in making this full publication of the methods now current in the design of large gas-holder frames.

The Cause of Rail Corrugation.

For a dozen years or more, foreign technical journals and proceedings of foreign engineering societies have been replete with articles on corrugated rails, or "roaring" rails as they are sometimes called. Within a comparatively recent time similar trouble has been experienced on some railways in the United States. We print on another page of this issue a paper by Mr. Chas. K. Mohler discussing the causes which produce corrugations on rails which will be read with interest by every engineer who has had trouble of this sort. Mr. Mohler's explanation will, we believe, appeal to practical men. He reaches the conclusion that corrugated rails are due to side slip of the wheels on the rail in combination usually with longitudinal slips. Every one is familiar with the way in which a heavily loaded surface will sometimes "chatter" when moving in contact with another, without lubrication. A good example is the chatter of brake shoes on a wheel under certain conditions. It seems entirely reasonable to believe that the side slip of the wheel on the rail which occurs in rounding a curve may take place with a certain chatter, and that this chattering sliding may start the corrugated wear of the rail.

In contrast with the common-sense treatment of the subject by our contributor, we are led to place a discussion of the same subject, which has just appeared in our foreign exchanges in the shape of a paper read before the British Association for the Advancement of Science. The engineering section of this Association usually obtains some very excellent papers summarizing recent progress in engineering. This paper in question is by Mr. W. W. Beaumont, M. Inst. C. E., a well known British engineer and investigator. Mr. Beaumont begins his paper by reviewing the literature upon the subject, which is indeed voluminous on the other side of the water. He then states that many engineers are still undecided as to the cause of rail corrugations and proceeds to give his own solution, remarking that the subject needs "the considered reasoning conjointly of the mechanic and the physicist." We print the beginning of Mr. Beaumont's explanation as follows:

> In a surface of a homogeneous solid under compressive stress in orthogonal directions in the plane of the surface, every particle tends to cause every particle surrounding it to recede in every direction. If one surface of a free flat bar of steel or iron be rolled or hammered, the material of that surface is subjected to a compressive stress. Of this stress it is relieved in the direction of length of the bar so far as the stress is no greater than that which is necessary to bend the bar, and cause it to keep the bent form. If of wrought iron, the molecular surface stress will in part be relieved by flow of the material or detrusion along the edges of the hammered surface, and the longitudinal bending be less than if the bar be of rail steel, in which the detrusion will be less.

We shall not weary our readers with a longer extract from Mr. Beaumont's scientific discussion of the subject, but will say at once that after several columns of this sort of argument he states his conclusion, which is briefly that rail corrugation is due to the fact that the pressure of the wheels on the rails exceeds the limit of elasticity of the rail steel! He proves this, to his own satisfaction at least, by measuring the contact areas of a car wheel resting on a rail, there being a piece of tissue paper and thin carbon paper placed between the points of contact. Assuming that his hypothesis is correct, Mr. Beaumont says that the way to stop corrugation is to lessen the wheel loads, to use larger diameter wheels, to run at lower speeds and to use harder rails.

It is safe to say, however, that none of these remedies will be adopted in practical railroad operation. The rolling stock of American steam railways at the present time has considerably higher wheel loads than are used in any other country of the world, yet very little trouble with corrugated rails has been recorded on American steam railways.

Mr. Beaumont's own statement of the conditions under which corrugation occurs points directly to the correctness of the conclusions reached by our own contributor, Mr. Mohler. Corruga-

tion is most prevalent on long radius curves. It is on such curves that side slip with a chattering effect is most likely to take place. On curves of short radius the slip is so great as to be continuous.

As to how this annoying rail wear can be remedied, that is another story. The best suggestion that we can make is that the railways give encouragement to a really scientific treatment of the problems of car truck design and curve mechanics. It is quite possible that the radical innovation in railway car trucks designed by Mr. Gustav Lindenthal a few years ago might so reduce the rail wear on curves and lessen the slipping and jerking as to remove the troubles from rail corrugation.

The Approaching Legal and Physical Limits to Sewage Disposal by Dilution at Chicago.

It will be twelve years on Jan. 1, 1912, since sewage purification by dilution, in accordance with well-considered engineering plans, was put in use by the Trustees of the Sanitary District of Chicago. In another ten years, or by 1922, it is estimated that the population of the District will have reached 3,000,000, and thus the total available amount of diluting water which the Federal Government has authorized withdrawn from Lake Michigan will be required to meet the statutory provision for a dilution of 3⅓ cu. ft. per sec. for each 1,000 population contributing sewage to the Canal.

Barring certain unforeseen conditions, which were perhaps quite excusable at the time and which do not effect the main principle involved, sewage disposal by dilution at Chicago has been a success and exclusive reliance upon it might be continued for an indefinite period if the volume of diluting water and the capacity of the Canal to carry it could be increased to keep pace with the growth of population. Under existing conditions, however, it will become necessary to reduce the volume of organic matter which will be contributed to the Canal by 1922 if danger of serious nuisance is to be avoided. This can be done by removing a small percentage of the organic matter from all the sewage or a large percentage from a portion of it, the total reduction to be the same in either case. Economy of both capital outlay and annual expenditure point to a relatively high degree of reduction of organic matter for a portion of the sewage as the better of these alternatives.

In the light of present information, settling and sludge-reducing tanks of the Imhoff type appear to be advisable under either of the two alternative plans, with probable dependence upon the tanks alone for relatively slight treatment of all the sewage, or upon the tanks followed by sprinkling (percolating) filters and finally by secondary Imhoff tanks for the more complete treatment of a smaller portion of the sewage. In any event, a considerable amount of purification would be required by 1935, and this would probably be beyond the possibilities of sedimentation alone.

In view of these opinions it is considered advisable that plans be made at once for treating portions of the sewage of Chicago, with a view of the early construction of works at points where they are needed most and of following this construction with further works as occasion may demand.

Intercepting sewers to divert from Lake Michigan the sewage of the Calumet section at one extreme and of the Evanston section at another extreme of the Sanitary District are recommended, with Imhoff tanks in each case. Until the Calumet interceptor is available, the 68th St. portion of the Chicago water-supply should be treated with hypochlorite of calcium, beginning this coming winter. This is necessary because a flood in the Calumet, accompanied by a southerly wind, pollutes water drawn from the 68th St. crib.

Such, in brief, are the leading conclusions and recommendations which have resulted from a detailed study made by Mr. G. M. Wisner, Chief Engineer of the Sanitary District of Chicago, and which are embodied in a report which we have abstracted at length elsewhere in this issue.

Those interested in the refinements of sewage disposal will wish to study our abstract in detail. Those whose interest is chiefly in the broader sanitary and economic aspects of water pollution and its remedies will perhaps find the preceding summary and the following discussion of a few points suggested by the report quite sufficient for all their purposes.

First of all, we wish to call attention to Mr. Wisner's conclusion that sewage disposal by dilution has been a success at Chicago and to urge that his recommendation that plans be made at once for a comprehensive system of sewage treatment implies no failure of sewage disposal by dilution. As we have already intimated, the long and short of the matter is that the rapid growth of Chicago, combined with the limit set by the Federal Government upon the amount of diluting water which can be diverted from Lake Michigan, render it necessary to make ready to lighten the burden on dilution by removing some of the organic matter from the sewage before it is discharged into the Drainage Canal and its various tributaries. This is no more a condemnation of sewage disposal by dilution than the necessity to build a second aqueduct to meet increasing water consumption would be a condemnation of a source of water-supply. Each is merely an example of the exhaustion of capacity. If any criticism could be made in either case it would lie in failure to foresee all the conditions that would make the demand exceed the supply.

In the case of the Drainage Canal, few considerations other than a more rapid increase in population, or a lower limit on volume of diluting water, than had been expected have played a part in the exhaustion of the diluting capacity of the Canal. These other considerations, or the principal ones, have been: (1) a possibly too low factor of dilution; (2) basing the factor of dilution wholly or chiefly on population, to the neglect or relative neglect of the immense volumes of industrial wastes discharged into the sewers or directly into the various channels which receive the sewage, and (3) various possible nuisances, more or less local in character due, to overdoses of sewage at single points and to sludge deposits which might occur not only at those points, but over long stretches of the Canal.

As to the factor of dilution, it should be remembered that when the original studies for the Drainage Canal were made, beginning some 25 years ago, much less was known about sewage disposal than now, particularly as regards disposal by dilution. On referring to Mr. G. P. Brown's "Drainage Channel and Waterway" (Chicago, 1894), we find a report made by a commission appointed by the Mayor of Chicago in 1886. This report was made and signed by Messrs. Rudolph Hering as chief engineer, and Benezette Williams and Samuel G. Artingstall as consulting engineers.

On page 361 of the volume named, we find the following in regard to dilution:

> For the purpose of estimating the cost of the water channel we have assumed 3,600 sq. ft. for the cross-section and a velocity of the water 3 ft. per sec. or 2 mi. per hr. This gives a discharge of 600,000 cu. ft. of water per min., or 24,000 cu. ft. for each 100,000 persons, which we believe equal to the maximum requirements of a population of 2,500,000 people.

A little calculation will show that the factor of dilution, as now commonly expressed, would work out from these figures at 4 cu. ft. per sec. of diluting water per 1,000 population, instead of the 3 1-3 ft. factor which was afterwards decided upon and embodied in the Illinois statute authorizing the construction of the Drainage Canal. We mention this in passing as a matter of justice to Mr. Hering, who for many years past has been credited with having established the 3 1-3 dilution factor. Had a factor of 4 been adopted, as originally contemplated, it would have made at least theoretically possible a postponement of the time when it would be necessary to supplement disposal by dilution at Chicago by some method of sewage treatment. We say "theoretically possible" because such practical matters as the limit put upon the diluting water diverted from Lake Michigan by the Federal Government reali[z]e it quite immaterial whether the facto[r] 3⅓ or 4. Notwithstanding this, the exact [recom]mendation made by Mr. Hering should be [bor]ne in mind.

As noted in Mr. Wisner's repo[rt] exhaustion of the diluting capacit[y] of the Drainage Canal (and we presume, so[me] of the local nuisances as well) is due in co[n]siderable measure to the enormous volume of h[ea]vy putrescible organic matter discharged into [the] Canal and its branches by various industri[es] and particularly by the stockyards. In som[e] respects it perhaps even more difficult [to] forecast the growth of industrial wastes th[an] of domestic sewage tributary to any sewage [sy]stem. Moreover, an engineer in designing a sewerage and sewage-disposal system is, to some extent, justified in assuming that if an abnormal increase in manufacturing wastes occurs, a portion at least of the responsibility of taking care of them rests upon the manufacturers. Experience in other cities indicates that possibly some of this burden may yet be put upon industrial concerns at Chicago, possibly to their economic advantage—but this it outside of the present discussion.

We judge from Mr. Wisner's report that, as might be expected under such complicated conditions as exist at Chicago, strictly local nuisances have developed at some points along the Drainage Canal and its various tributaries. We are not at all sure that these could be wholly avoided at Chicago by any reasonable system of sewage disposal by dilution; or, in other words, a partial purification might be required to meet special but relatively minor local conditions without at all impairing the general principal and practice of sewage disposal by dilution.

Some of the local nuisances which we have in mind appear to be caused largely, if not chiefly, by sludge deposits. Some of these deposits, it would appear, are so extensive in area, or length of channel, as to become almost general rather than local. These appertain particularly to the larger or earth sections of the Canal and to the vicinity of the controlling works, in both of which cases the velocity of flow is diminished to such an extent, it appears, as to result in a deposition of sludge beyond the capacity of the diluting water to assimilate or pass along without nuisance from septic action and evolved gases. In other words, portions of the Drainage Canal (we use that term broadly to include the Canal and all its tributaries) appear to have been converted into virtual elongated septic tanks during the past summer. Low water and hot weather appear to have contributed materially to this undesirable condition, but as these are likely to occur again, either singly or combined, the[y] are future contingencies which must be reckoned with.

These sludge deposits naturally suggest the limitation which may be put upon sewage disposal by dilution at many points in the country where otherwise the system would be wholly satisfactory; in other words, they point to the frequent necessity of supplementing disposal by dilution, or of lightening the burden of that system by removing a greater or less portion of the suspended matter from the sewage before it is discharged into the diluting water. Unless this necessity involves too much treatment the system employed may still be called disposal by dilution.

It will be remembered that the object of the Chicago Drainage Canal was two-fold: (1) the protection of the water-supply of Chicago by diverting the sewage from Lake Michigan to the Des Plaines River and thus from the Illinois to the Mississippi, and (2) the remedying of the intolerable odors and hardly less tolerable sights which had resulted from the discharge of a large part of the sewage of the city into the Chicago River, which stream for the most part was very sluggish and thus gave rise to extremely offensive conditions. It appears from Mr. Wisner's report, and also from information which comes to us from other sources, that as regards the protection of the water-supply of Chicago, the Drainage Canal has been notably

successful thus far and will be still more when it is completed in all its parts.

There yet remains occasional marked danger to that portion of the Chicago water-supply which is drawn from the 68th St. intake. The sewage of a rapidly growing section is being discharged into the Calumet River and at times when flood waters are "accompanied by a southerly wind," sewage pollution is borne to the intake crib. The proposed Calumet-Sag Canal, combined with an intercepting sewer along the Calumet River, will remove this danger. Until this Canal and intercepter are built, there is danger of occasional serious pollution. On this account Mr. Wisner recommends the immediate installation of a hypochlorite plant for use in treating the water drawn from the 68th St. crib in case of such emergencies as have just been described. He also suggests the ultimate possible need of filtration for this source of supply.

We think it pertinent to ask whether, in the not far distant future, Chicago will not find it advisable to purify its entire water-supply. Considerable volumes of polluting matter are always bound to be discharged into Lake Michigan in times of flood by both the Chicago and Calumet Rivers, even though the sewage of the city be diverted southward. While we have at the present moment no definite information on the point, we assume that in times of severe storm most or all of the 156 sewer outlets mentioned in Mr. Wisner's report must send some of the excess storm water by its original course into Lake Michigan. Both rivers are also more or less polluted by other means. Although the water intakes are a considerable distance from the shore, sight of the possibility of occasional contamination should never be lost.

Moreover, there will always exist more or less danger from contamination by shipping and by the dumping, near the intakes, of dredged matter by tugs—of which the possibilities are pointed out by Mr. Wisner, with recommendations for precautionary measures. Rigid as these precautionary measures may be in design and execution, there will always exist the possibility of contamination from shipping and from deposits of dredged matter, and also the possibility of pollution from the rivers. Beyond all this, it should be remembered, as we have often pointed out, that filtration of all water from surface sources is likely to be insisted upon in this country before many years, as is the case in those European countries which stand in the forefront of water sanitation.

In saying these things it should be understood that we dot not mean to urge that refinements in water-supply be undertaken while other and more important protection of the public health go by the board for lack of funds.

Finally, those responsible for authorizing Mr. Wisner's investigation and report, and also Mr. Wisner himself and his associates, deserve commendation for beginning to plan to meet the sewage-disposal needs of Chicago years in advance of the time when action will become necessary. It is only by far-seeing studies of this character that Chicago or any other of our large cities, with their rapidly-growing population and constantly changing conditions, can safeguard themselves from sanitary or economic disaster. The time is past for the haphazard disposal of sewage. Hereafter, no city which has not already met the problem should lose time in making ample provision for all future contingencies that can be foreseen by the wisest engineers. There is, of course, much that cannot be foreseen. This makes it all the more necessary that such plans as are formulated should be of such a character as to be readily adaptable to any remote contingencies that may arise. So far as the Chicago Sanitary District is concerned, it should, of course, be understood that the studies outlined in Mr. Wisner's report are only preliminary to the more detailed plans which should be made. It is particularly desirable that he and his associates should be permitted to continue their studies of certain special means of treating both sewage and sludge mentioned in Mr. Wisner's report, including rapid filtration and aeration of sewage and incineration of sludge.

LETTERS TO THE EDITOR.

Hydrostatic Pressure Beneath Dams.

Sir: In referring to Mr. Wegmann's article on masonry dams, in your issue of Nov. 16, 1911, p. 594, regarding hydraulic uplift, I beg to submit the following data taken from the Volume on Water Power in the Handbuch der Ing. Wissenschaften, 1908 edition.

In the design of the dam at Marklissa across the River Queis in Prussia, completed in 1904, the designer, Engineer Intze, considered the full static pressure of the water acting underneath with undiminished force from the upstream face to the downstream face or toe of the dam. It is stated that on account of the excellent workmanship this condition can never arise, but was assumed in order to allay the fears of the citizens living below the dam. This dam has a width at the crown of 5.7 m., a height of 43 m., a base of 37.7 m., or about 87% of the height, and contains 65,000 cu. m. of masonry.

The European engineer, Fecht, recommends (Zeitschrift f. Bauwesen, 1889) that the full hydrostatic pressure be assumed as acting underneath at the upstream face and diminish to zero at the downstream face or toe of the dam.

G. M. Braune.

Birmingham, Ala., Nov. 20, 1911.

Sir: The article by Mr. Edward Wegmann in your issue of Nov. 16, 1911, p. 594, on under pressure on masonry dams should not go unchallenged. The arguments by which Mr. Wegmann would lull engineers to sleep until the next periodic dam failure are exceedingly weak. Let us analyze them.

Argument One: Mr. Fteley considered that water passes through the masonry by capillary action. The Flood Commission in Pittsburgh, in their investigations, found that all over the downtown portions of the city water rose in basements of buildings to exactly the same height—exactly the height of the crest of the flood. Did capillary action carry that water thousands of feet through compact earth? And did it lift thousands of tons of it into these basements without pressure? Tests have shown that when concrete is laid on bed rock the water pressure at the surface of that rock as shown by a gage will correspond to the full head.

Arugment Two: upward pressure under the base of a dam or at any assumed horizontal joint would only exist, if at all, over a very small area of the whole base or joint. Facts are against this—the facts of awful wrecks, which **could not have happened if this pressure were exerted only "over a very small area of the whole base."**

Argument Three: close inspection will fail to detect any sign of incipient failure in certain dams where upward pressure was neglected in their design. This is the old argument that a structure is "giving satisfaction." A girder span in Allegheny gave satisfaction for 30 years or more and showed no signs of failure; it then failed. A roof stood up for 42 years without showing signs of failure, and then collapsed seven years after it had been condemned by an engineer. (See Engineering News, Dec. 19, 1907, p. 681.) Among the examples cited where ice pressure and upward pressure were neglected the New Croton Dam is given. Whatever may have been the designer's modus operandi, it is not true that that dam would not be completely stable under full upward pressure. The dam is 297 ft. high and 206 ft. wide at the base. It is 18 ft. wide at the top. Furthermore, an ample spillway prevents water from reaching the crest by a very substantial allowance. If all dams were built with the same disregard (?) of upward pressure we would not read of a failure every few months.

Argument Four: "If the last-mentioned dam had been designed to resist a full upward pressure under its base and an ice pressure of 40,000 lbs. per lin. ft. of dam, its base would have been over 400 ft. wide, instead of 206 ft., as actually built." Why cloud the issue with statements like this. Ice pressure, if it could exist in any such amounts, would be a local pressure. It would break out the crest of the masonry or crack it and then have spent its force. Heavy expansive force on the top of a masonry pier would probably unseat the coping, especially if little or no weight rested upon it. There is no occasion to couple ice pressure, that may exist in rare occasions in peculiar circumstances and do local damage, with vertical pressure in horizontal joints, that is apt to reach its full value in almost any location or condition. and which, if not provided for, will wreck the entire structure. It savors of subterfuge to say that because the same engineer suggested that both ice pressure and under pressure should be allowed for, both suggestions must be adopted or rejected together. Whatever virtue there may be in the consideration of ice pressure (and it is unquestionably a desirable precaution), it has absolutely nothing to do with consideration of the natural and constant pressure of water in a structure to impound water.

Argument Five: "Engineers will readily agree that where such great interests are at stake, it is justifiable to make assurance doubly sure by including in the calculations of the profile of a masonry dam every conceivable force that might contribute to the failure of the structure. But is it the best practice to adopt this method generally?" What is the limit of population that it is worth while taking precautions not to murder? Is a small industrial town too insignificant to make it worth while to spend a few more thousands of a client's money to "make assurance doubly sure?" Is not a poor farmer with his little family, or even a single hermit, entitled to life, liberty and the pursuit of happiness that may be jeopardized by parsimonious economy on the part of capitalists who build dams, and who frequently lose hundreds of thousands of dollars because of this supposed economy?

Argument Six: Mr. Wegmann gives a short list of dams over 50 ft. high that have failed and the alleged causes of their failure. Why the 50-ft. limit? Thirty or forty solid masonry dams have failed in the last 21 years. There is no doubt whatever that a large number of these were lifted and lubricated by water underneath them and then pushed out by the horizontal force of the water.

Argument Seven: "Because two or three masonry dams built on shallow foundations of pervious rock have failed, shall we hereafter assume in our calculations that the dams we are designing are to stand on such foundations?" Legislation demands that buildings be designed for dead and live loads. Buildings very seldom receive their full live load; some of them never do. Because two or three buildings have failed when a lot of people unduly crowd on the floors, shall we assume that buildings we are designing are to stand such treatment? Legislation ought to demand that dams be designed for the upward pressure of water beneath them. It is more probable to occur on any kind of foundation than the full live load of almost any building or part of a building. It is just as great a crime to ignore it in designing as to ignore live or dead load in a structure.

Edward Godfrey.

Monongahela Bank Bldg., Pittsburgh, Pa., Nov. 18, 1911.

[Note the article on this same subject entitled "Provision for Uplift and Ice Pressure in Designing Masonry Dams," by Mr. C. L. Harrison, M. Am. Soc. C. E., reprinted on p. 663 of this issue.—Ed.]

The Offer of the Office of Public Roads to Do Free Bridge Engineering.

Sir: You can render no better service to the engineering profession than such editorials and articles of yours as contained on pages 600 and 601 in Engineering News of Nov. 16 under the caption "An Offer of Free Highway Bridge Engineering Work by a Federal Bureau."

The action taken in this matter is so radical and is such a blow at the engineering profession that your editorial comment is fully justified and should be applauded by every engineer in the country.

It would be pertinent to know (and I would suggest that you obtain the information) under what act of Congress the office of Public Roads was established and whether or not the action of the Director is justified by the powers expressed in the act; also under what particular appropriation of Congress are the expenses of engineers for special localities provided.

John F. Wallace,

10 Bridge St., New York City, Nov. 21, 1911.

Sir: Anent your article, on the furnishing of "Free Bridge Engineering" by the Office of Public Roads (United States Department of Agriculture), in your issue of November 16, permit the writer to express his appreciation of your attitude and remarks.

The writer is not a consulting engineer in active practice at present. If he were, he would feel inclined to do more than to merely write this letter. However, in the interest of consulting engineers, of chemists, and of professional men generally perhaps, the writer wishes to let you know of his agreement with your remarks referred to,

and to suggest that they should be made even stronger and even more comprehensive. Enclosed is a poster showing another offer of "free" assistance by the Office of Public Roads, which in the opinion of the writer, is as reprehensible in principle, at least, as the one you condemn. Certainly a pertinent query for professional men to ask now is "Where is all this 'free help' idea going to lead to?"

In the opinion of the writer, the Office of Public Roads has a sufficient field for valuable work in road matters without encroaching on the fields of the various professions. The writer has only the kindest feeling toward the Office for its future, as well as from its past, provided it confines its activities within proper bounds.

Roads Engineer.

Baltimore, Md., Nov. 19, 1911.

[The poster enclosed advertises a "Good Roads" meeting held at Elkton, Md., Nov. 11, in connection with the proposal to build a "good road" in the locality "at a cost not to exceed $3,000 a mile." The poster states further that "The United States Road Commission will make the survey free of cost and give the services of engineers to superintend the construction."—Ed.]

Sir: I certainly agree with you that Bulletin No. 39 of the Office of Public Roads concerns an important subject, and especially so to the engineer practicing in rural districts like the one where I am situated.

We have fought here for some time against the bridge companies' offers to design bridges for the county and town authorities, free of charge, making all necessary surveys, etc., in connection with their plans, thus making it difficult for the independent engineer to practice that part of his profession.

The Government's proposal to make the necessary surveys and designs for bridges and culverts practically free of charge seems to me to be putting a stumbling block in the way of our profession.

The bridge companies, of course, use their designs for the purpose of securing contracts, whereas the Government has nothing to gain, except it be a possible safety factor to the public.

But there are ample checks upon any engineer's design in this state, in that he is required to have his plans passed upon by the County Superintendent of Highways, who is undoubtedly capable of advising whether or not the bridge is properly designed. So, in my opinion, the Government is wrong in discouraging the private practice of bridge engineering and making it harder for many an engineer to obtain a living.

I think this matter should be discussed among engineers, and certainly some action ought to be taken in reference to it.

A. R. Applegarth, C. E.

Patchogue, N. Y., Nov. 17, 1911.

Sir: Referring to the article in Engineering News of Nov. 16, entitled "An Offer of Free Highway Bridge Engineering Work by a Federal Bureau," allow me to register my protest against such extension of Federal activities. Not only is it unnecessary and fatal to the independent engineers in that line, but it is manifestly unjust to distribute what must inevitably be a high engineering charge, over the entire jurisdiction of the Federal government, when the service is rendered to one county of one state, and is not at all a matter of national benefit.

In support of the new bureau, the theory must be that the recognized need of good roads is so nearly universal that it is national. And no doubt the improvement of highways tends to increase national prosperity. But the time is past when a highway is an important **interstate** artery of transportation, to be controlled by the Federal government. The highway of to-day is a feeder for railway and steamship lines, primarily, and our state, county and town organizations have ample power, as well as paramount interest, in the improvement of roads and bridges.

The requirement of the Post Office Department that good roads must be provided for rural free delivery routes is just to all sections of the country, and has contributed to the improvement of many miles of road by stimulating local interest in the matter. Thus far Federal encouragement is a help to those who help themselves.

As many engineers, at least in the West, are aware, the Federal bureaus not only offers, but have for years furnished, free engineering work for drainage projects, reclaiming overflowed lands along minor streams. One case, in which I was particularly interested because the Federal engineers took a piece of work which I would otherwise have done, was in southeastern Nebraska.

The Nebraska Legislature in April, 1909, passed a very good law authorizing land owners along any swamp or overflowed lands in the state to organize a drainage district, and provided a method of assessing the cost among the owners of the land actually benefited. The organization was to be permanent, and was authorized to employ all necessary legal, engineering and clerical assistance as was deemed best.

In the case referred to above, several engineers, including myself, were present at the first meeting of the land owners, and each made his proposition to the assembled farmers. At the second meeting, one man announced that it would not be necessary to employ an engineer, as he had a letter from a government official stating that the Bureau of Drainage Investigations would do the engineering for nothing, local expenses only being paid by the land owners. This statement, coming just after the chairman had told me that I was selected as engineer, was a hard pill to take, and especially so, when, about three months later, I saw three boys just out of school doing the field work, their pay coming from Washington.

I have thought the matter over a great deal since that time, and still fail to see how that drainage in Nebraska could benefit persons in Maine, who pay their share of the internal revenue.

I wish to thank Engineering News for this as well as other movements in behalf of the profession that does things, and I am to-day placing the matter before the Engineer's Society of which I am a member, and also before the Congressman from my district.

Ira W. Dye.

4021 West 18th St., Chicago, Ill., Nov. 18, 1911.

Sir: I have read with great interest the editorial remarks in this week's Engineering News on the proposition of the Office of Public Roads to make, free of charge, plans for highway bridges.

You will recall that a few years since you eliminated from an article prepared by me some criticism of the various governmental bureaus for magnifying their importance and overstepping the bounds of their proper spheres. I have felt very strongly about this matter for years and am glad to see that the practice has now reached a point where you openly condemn it.

While this particular instance is a little more flagrant than usual, it is only the logical outcome of policies which have been pursued for many years by the Washington departments and bureaus. In fact, the office of Public Roads has for years offered to send its employees to any part of the country to design and assist in the construction of roads.

Unless they receive a decided check from some quarter there is practically no limit to which these bureau ambitions may not be expected to go.

In our former correspondence you will recollect that I called attention to such wholly unwarranted action as the issuance by the Forestry Division of a bulletin on wood block paving and the undertaking of the Geological Survey to conduct experiments on the value of fuels, tests of water wheels, etc., all of which are wholly foreign to the legitimate work of these departments.

I have long felt that such activities by Federal bureaus are wholly wrong and ought to be stopped. If the present incident serves to show how indefensible the practice is, and to arouse proper opposition to it, I shall consider it a fortunate occurrence.

While it must be conceded that the Good Roads Division of the Agricultural Department has done a lot of good work, it may be questioned whether its object and practice does not violate the Constitution of the United States. The Federal government derives its whole authority to undertake public improvements from that clause of the Constitution empowering it to regulate commerce between the states. In nearly every case the roads dealt with are purely intrastate and have no relation whatever to interstate commerce.

It was the custom of the Presidents up to the Civil War to veto such measures for purely State improvements when Congress ventured to pass appropriations for them, and it seems to me they were right. If the National government may build local roads wholly within a State, it may with impunity undertake any other local improvements wholly within and for the benefit of a single State. The Constitution certainly never contemplated that this should be done.

S. W. H.

New York, Nov. 17, 1911.

Sir: I desire to offer my modest commendation for your prompt, timely and forceful editorial on "An Offer of Free Highway Bridge Engineering Work by a Federal Bureau." I received one of those circulars, and after reading it was aghast, though some projects are unworthy of more than laughter.

What next?

Has the nation become so illiterate and the profession of engineering so senile that central Federal bureaus must be created to do at no cost to a builder of bridges but at an exorbitant cost to the whole public professional technical work? Mr. Ballinger, late Secretary of the Interior, was indeed a prophet (though without honor) when he said that Bureaucracy is the dream of certain political effluvia. A few more such propositions, and, presto, we shall have a socialistic Utopia leading to the dead level of National decay. The theory involved is the very antithesis of the theory of individuality and is antagonistic to the theory of our industrial, commercial, professional and artistic life.

Highway bridge engineering, or any other engineering, needs no such certified proprietary. Between up-lift politics, bureaus, strangulation and ham-string refinements, now the order of the day, we all might well and profitably melt, thaw and resolve ourselves into some sorely-needed dew.

J. C. Ralston.

Spokane, Wash., Nov. 21, 1911.

Sir: Your editorial on the subject of free bridge designing by the Federal Government, meets with my approval. It may interest you to know that I lost no time in communicating with the Congressmen and Senators from this state and have also sent communications to some engineering societies of which I am a member. Other engineers have told me they intend doing likewise.

We realize, of course, that it is only a question of a short time when the engineering activities of the Federal Government will become so large that a government school of engineering must be established, like the famous Ecole des Ponts et Chaussees of France. Students will be admitted after severe competitive examinations and be given free tuition, together with an annual salary of from $500 to $600 per year, as at West Point and Annapolis, with which to purchase books, and feed and clothe themselves. In return for this they must pledge themselves to work for the government for a period of five years after graduation, in any capacity and for low salary, just as army and naval officers do. If they wish they can have life positions and be retired after thirty years' continuous service with a liberal pension, to make up for the low pay. From such a school the irrigation and reclamation departments, highway bureaus, quartermasters and engineer corps can obtain assistants.

Until such a school is established, however, it is unfair for engineers in the employ of the Government to lend themselves to any such proposal as that set forth in Bulletin No. 39, Office of Public Roads. It is highly unprofessional to say the least.

Severe competition must be faced by all engineers in private practice and this legitimate competition is hard enough without the Federal Government joining the big corporations in the giving of free advice and making presents of plans and inspection services. It will be only a short time under such conditions when American engineers will be in the position of British engineers, glad to secure work at a compensation of $10 or $20 per week. Private practice will be left to men who by fortuitous circumstances rise to a post where the Government will pay them a retainer large enough to enable them to maintain an office and with the prestige thus afforded, get free advertising.

At present the engineering courses in all colleges and universities are the most expensive courses in such institutions. A professor in one of these schools has the reputation of being the mostly highly paid engineering professor in the country. In addition it is popularly believed that his income from textbooks prepared by him is the equivalent of an ordinary living salary. It is also currently believed that his per diem charge when employed as a consulting engineer is $500 plus all expenses. To the average engineer it looks as if this man had enough money and fame to be content and not to stoop to compete with engineers engaged in a small practice.

Ernest McCullough.

Monadnock Block, Chicago, Ill., Nov. 24, 1911.

Provision for Uplift and Ice Pressure in Designing Masonry Dams.*

By C. L. HARRISON,† M. Am. Soc. C. E.

There has been much discussion recently by engineers and the technical press on the upward pressure of water and ice thrust in dams. The following brief statement is written to bring the subject before the Society in the hope that it will be fully discussed.

Uplift.

For convenience in discussing this subject, reference is made particularly to masonry dams on rock foundations. The principles involved will apply equally to other foundations and to dams built of other materials. The upward pressure may be due to water getting into the foundation of the dam or into the dam itself.

Foundations vary so much in character, that it is necessary to study each particular site before deciding to what extent water may get into them.

(1) In the case of a foundation of hard, sound rock, without either horizontal or vertical seams, there is no reason to expect that water will get into it and produce an upward pressure, and, in the design, no allowance should be made for it. In such cases the junction between the masonry and the foundation can easily be made watertight.

(2) In the case where the foundation is stratified with well-defined horizontal seams, and the dam is located near a fall or rapids in the stream, so that the water may flow from the seams at the toe of the dam as freely as it enters them from the reservoir, the upward pressure will be approximately equal to the static head at the heel and gradually decrease to zero at the toe of the dam.

(3) Take a foundation similar to the foregoing in every respect except that the water in the seams of the rock cannot escape freely near the toe of the dam, but must flow some distance down stream through rock or other materials before it reaches the surface of the ground, or must rise vertically to the surface: Then the upward pressure at the heel will be equal to the static head, and that at the toe will be equal to the head required to overcome the resistance to the water escaping at that point.

While these three cases present well-defined conditions, it is probable that at most sites the conditions will lie between those presented in Case 1 and in Cases 2 and 3, that is, the water will not be in the foundation throughout its entire area, but will cover only a part of this area. This makes it necessary to study the foundation carefully at each site in order to determine to what extent water may get into it. When this upward pressure exists, weight must be added to the dam by additional masonry to counterbalance it. Generally, it will be found cheaper to make large expenditures to provide a cut-off in the foundation, which will not only reduce the uplift, but will also save the water. Such a cut-off should be located at the heel of the dam. If it is located under the middle of the dam, there would be an upward pressure under the upstream half of the dam, due to the full head of the water in the reservoir.

A thorough investigation, by borings and otherwise, should be made of the foundation at each site before the dam is designed, and a liberal margin should be allowed over what the engineer (basing his figures on his experience and best judgment) believes to be safe.

In order to determine what allowance to make for pressures due to water which gets into the dam itself, one must first decide on the character of the construction. With suitable stone, sand, and cement, it is possible to build a masonry dam which will have no horizontal cracks or seams, and it is also possible to provide against vertical cracks, to a large extent, by expansion joints. Water in vertical cracks, however, does not produce an upward pressure. In such structures very little, if any, allowance should be made for the upward pressure due to water getting into the masonry.

If the materials for building water-tight masonry are not to be had at the site of the dam, and it is very expensive to import them, it is generally advisable to adopt a different class of masonry, which will probably be more pervious and also more difficult to construct without horizontal cracks or seams, thus allowing the water to enter the dam, and resulting in upward pressures. The extent of such pressures will depend on the character of the masonry and the care with which it is built, all of which must be known before an estimate can be made of the extent to which the water will get into the dam. The effect of this upward pressure, however, must be counteracted, either by increasing the section of the dam or by increasing its height above the water level in the reservoir, or by both. In many cases it may be advisable to provide drainage wells near the upstream face to intercept the water and carry it off through pipes at the toe of the dam, thus reducing or eliminating its effect in the main body of the dam. After determining the type of masonry to be constructed, it is still a question of judgment, based on observation, tests and experience, as to what the upward pressure in the dam will be.

The upward pressures in the foundation, and in the dam itself, should be considered separately before a decision is reached.

Ice Pressure.

After ice has formed on a reservoir, it contracts under a lower temperature and expands under a higher temperature. The contraction due to the cold weather of winter results in cracks which fill with water that freezes and produces a continuous sheet of ice over the surface of the water. Under the higher temperatures of the late winter and early spring, the ice is warmed up and expands. On reservoirs this results in the ice being forced up the banks, unless the inflow of water should raise the level sufficiently to provide the increased area. In case the banks are vertical or nearly so, the expansion produces a pressure on the sides of the reservoir and results in the ice being compressed to some extent and buckling up out in the reservoir. In some cases the ice may be crushed.

In designing dams, the ice pressure should be considered, but it is of less importance than the upward pressure of water in the dam and its foundations. The ice is in sight and can be cut along the face of the dam, and thus relieve the pressure. Generally, in storage reservoirs, the period of heavy ice pressures is also that of low water, and the pressure would come against the dam at a point considerably below the high-water level, where the dam is strong enough to resist it. In many cases, the dam is located in a narrow gorge where the full effect of the ice field cannot reach it. The dam should be strong enough to resist the water pressure and the additional pressure caused by the ice, but what this will be depends on the thickness of the ice and other local conditions at each site, and no general rule can be made to cover all cases.

*Paper presented before Dec. 20, 1911, meeting of the American Society of Civil Engineers. Published in November, 1911, issue of the Proceedings of the Society, p. 1195.

†Engineer, 7 East 42d St., New York City.

Reinforced-Concrete School Buildings to the number of 231 are under construction in the Philippine Islands, according to a paper by Frank R. White, Director of Education, in the annual supplement to the Manila "Times." Mr. White says: "We have come to the conclusion that it is a waste of good money to construct school buildings of other than the most substantial type." The buildings are constructed on standard plans prepared for provincial, central and barrio schools. A unit system of building is adopted by which additional rooms may be added to the building as they may be required.

A Diesel Engine Ship of 10,000 Tons.—Messrs. Barclay, Curle & Co., Whiteinch, launched on Nov. 11, the twin-screw oil-engined vessel, "Jutlandia," for the East Asiatic Co., of Copenhagen. The vessel is 384 ft. in length, 53 ft. 3 ins. in breadth, 30 ft. in depth, 23 ft. 6 ins. draft, of 10,000 tons displacement. She will be supplied by the builders with two sets of Diesel oil engines capable of developing 3,000 indicated horsepower. The fumes from the engine room will be led up inside the mizzen mast and exhausted at a height of 48 ft. above the deck, so that a funnel is not required, even for exhaust purposes. The siren, on the main mast, will be operated by compressed air. The vessel will have no steam for mechanical purposes, as even the donkey engine will be an oil engine, and it will also generate the electricity which will operate the twelve cargo winches and the anchor windlass. The machinery space is only about a third of that which would have been necessary for steam engines, and the absence of boilers and boiler casings leaves a large amount of hold space for the storage of cargo, and far more deck space for the use of passengers than would be possible if the vessel were fitted with steam engines.—"Engineering," Nov. 17.

Operating Results of the Garbage-Reduction Works of Cleveland and Columbus, Ohio.

By JOHN H. GREGORY,* M. Am. Soc. C. E.

The question of garbage disposal is one which every municipality is compelled, sooner or later, to face. The solution in each case depends in a large measure on local conditions and can only be arrived at after careful investigation and study. In 1905 the writer was called on to make an investigation of the garbage-disposal question for the city of Columbus, Ohio, and in the report which was submitted he recommended the construction of a refuse incinerator, notwithstanding the fact that the amount of ashes produced in Columbus was small, due to the extensive use of natural gas as a fuel. It was appreciated that the amount of rubbish and ashes to be collected would not provide sufficient fuel for burning the garbage, and in the estimate of the cost of operation of the proposed works, an allowance was made for additional fuel for this purpose.

At the time this report was made the disposal of the garbage by the reduction process was seriously considered, but the writer, for several reasons, hesitated to recommend reduction, although the local situation was such that reduction would receive much more favorable consideration than in many places. With the exception of the works at Cleveland, all of the reduction works in the United States were owned and operated by private parties, and exact data as to the cost of operation of and income from these works could not be secured. The city of Cleveland had made public certain figures relating to the operation of the works there, but it was felt that these works had not been operated a sufficient length of time under municipal control to more than indicate in a general way what results could be obtained.

As with many municipal improvements, the wheels of progress turned slowly, and although funds were provided for the construction of garbage-disposal works, no definite decision as to the type of works to be built was reached until late in 1907. In the meantime, very satisfactory results had been obtained at Cleveland and the figures and data from these works were analyzed and examined with great care. The writer also had been afforded the opportunity of further visiting and inspecting many other garbage-reduction works. As a result of this further investigation and study he reached the conclusion that it would be perfectly feasible for the city of Columbus to build a garbage-reduction works and operate the same in a perfectly sanitary manner, without producing a nuisance, and further that the disposal of the garbage by reduction would be far more economical than by incineration.

As the time had arrived for reaching a definite decision as to the type of works to be built, the writer recommended the construction of a reduction works instead of a refuse incinerator, and as the city officials concurred in this recommendation, steps were taken which resulted in the construction of the municipal-reduction works at Columbus, which works have been in operation since July, 1910.

Where a modern refuse incinerator is installed, it has been the general practice to locate the same at some central point in the municipality, while on the other hand, with a reduction works, it has generally been the practice to locate such works at a distance from the municipality. It follows, then, that in order to deliver the garbage at a reduction works it would have to be transported an additional distance, causing thereby a charge for hauling or transportation in excess of that which would be required if the garbage was delivered at an incinerator, and in the analysis of the operating costs of both the Cleveland and Columbus works, the cost of transporting the garbage to the reduction works has been included.

In considering the financial aspects of the operation of a reduction works, or in fact any other type of works, capital charges must be included, made up of interest on the investment,

*Of Rudolph Hering and John H. Gregory, Consulting Engineers and Sanitary Experts, 170 Broadway, New York.

and also, either a sinking-fund charge or a charge for depreciation. Only by including such charges can a fair statement of the cost of operation be made.

Results at Cleveland.

The garbage-reduction works at Cleveland were purchased by the city of Cleveland from the Newburg Reduction Co., and have been operated by the city of Cleveland since Jan. 1, 1905. The reduction works are located at Willow, Ohio, some miles from the city, and the garbage is transported in cars from the city to the works, the garbage wagons delivering the garbage at a central point in the city where it is loaded into the cars. The works, since their purchase, have been improved and extended in many ways.

In Table I. is given a statement of the annual cost of operation of, and income from, the garbage-reduction works at Cleveland for the six years 1905 to 1910, inclusive. The table also gives the population, the total weight of garbage and the weight of garbage per capita per year. The figures giving the total cost of the works to the end of each current year do not include the cost of the cars used for transporting the garbage from the city to the works at Willow, nor do they include the cost of any work at the loading station where the garbage is loaded into the cars. In the Cleveland reports the transporting of the garbage from the loading station in the city to the reduction works is included as a part of the work of the Collection Department and the published figures are not in such detail that the cost of the cars for transporting the garbage or the cost of work at the loading station can be ascertained. This cost, under the conditions at Cleveland, however, compared with the cost of the reduction works, would be relatively small and its omission in the figures in Table I. does not materially affect the results given.

In the Cleveland reports of the cost of operation no interest charge is included, as the same is paid by the city out of general funds. In Table I., however, an interest charge of 4% per annum on the total cost of the works, as given at the end of each current year, has been included, and also the depreciation as charged off in the Cleveland accounts. Up to June 30, 1908, the depreciation on the buildings was charged off at the rate of 10% per annum, but since that date at 5% per annum. On the equipment the depreciation has been charged off at the rate of 10% per annum.

An examination of this table will show that for the first year, 1905, there was a net charge for operating the works, but that since 1905 there has been a net annual income, which income has steadily increased from year to year, amounting in 1910 to $72,532, or about $1.62 per ton of garbage handled.

Of the two products recovered at a garbage-reduction works, namely, grease and tankage, grease is the more valuable. The amount of grease which can be extracted from garbage, of course, depends on the character and composition of the garbage and on the details of the process carried out at the works. In Table II. is given the amount of grease which has been recovered at the reduction works at Cleveland for the last six years. It will be seen that the grease has varied from 2.63% of the weight of the garbage in 1905, to 3.75% in 1910. The increase in the percentage of grease recovered is due to the introduction of improved methods and machinery, and especially to the introduction of the percolating process, in which a solvent is used.

At Cleveland all of the tankage has not been dried, the relative amounts of dry tankage and pressed tankage having varied from year to year. On this account it is not possible to state exactly what percentage the try tankage would be of the garbage. It may, however, be approximated. Commercially dry tankage contains not over 10% of moisture. The moisture in pressed tankage varies, depending on the length of time the material is pressed and also on the type of press or method of pressing used.

It is perhaps fair to estimate that the amount of moisture in the pressed tankage at Cleveland would be about 60% of the total weight. In 1910, the pressed tankage amounted to 1,500 tons. Assuming this pressed tankage to contain say 60% of moisture and to be dried so as to contain only 10% of moisture, the resulting dry tankage would be 667 tons, which, if added to the actual dried tankage for the year, namely, 5,224 tons, would give a total weight of 5,891 tons of dry tankage. As the total weight of garbage handled in 1910 was 44,747 tons, the weight of dry tankage, had it all been dried, would therefore have been about 13.2% of the weight of the garbage.

The main income of a garbage-reduction works is derived from the sale of the two products, grease and tankage, the prices received for which vary ing from time to time, depending on the condition of the market. The prices which have been received for grease and dry tankage at the reduction works at Cleveland for the six years, 1905 to 1910, inclusive, are given in Table III. The average price received for grease was 3.8 cts. per lb. and for dry tankage $7.19 per ton.

TABLE II.—GREASE PRODUCED AT THE GARBAGE-REDUCTION WORKS AT CLEVELAND, OHIO, 1905-1910.

	1905.	1906.	1907.	1908.	1909.	1910.
Garbage, tons	30,382	34,891	37,606	41,247	44,590	44,747
Grease—						
Jan. 1-June 30, lbs.	756,833	1,024,895	1,225,290	1,416,785	1,[illegible]	1,865,789
July 1-Dec. 31 lbs.	841,430	1,115,380	1,140,980	1,423,084	1,[illegible]	1,490,295
Pounds	1,598,263	2,140,275	2,366,270	2,839,869	2,[illegible]	3,356,084
Tons	799	1,070	1,183	1,420	[illegible]	1,678
Year percentage, by weight, of garbage.	2.63	3.07	3.14	3.44	[illegible]	3.75

TABLE III.—PRICES RECEIVED FOR GREASE AND DRY TANKAGE AT THE GARBAGE-REDUCTION WORKS, CLEVELAND, OHIO, 1905-1910.

Period.	Grease per lb., cents.	Dry tankage, per ton.
Jan. 1, 1905-June 30, 1905	2.85	$7.08
July 1, 1905-Dec. 31, 1905	2.91	7.02
Jan. 1, 1906-June 30, 1906	3.02	8.42
July 1, 1906-Dec. 31, 1906	3.76	8.49
Jan. 1, 1907-June 30, 1907	4.25	7.85
July 1, 1907-Dec. 31, 1907	4.30	6.89
Jan. 1, 1908-June 30, 1908	4.35	6.71
July 1, 1908-Dec. 31, 1908	3.99	5.89
Jan. 1, 1909-June 30, 1909	3.46	7.68
July 1, 1909-Dec. 31, 1909	3.95	6.81
Jan. 1, 1910-June 30, 1910	3.98	6.98
July 1, 1910-Dec. 31, 1910	4.72	6.42
Average	3.80	$7.19

TABLE I.—ANNUAL COST OF OPERATION OF AND INCOME FROM THE GARBAGE-REDUCTION WORKS AT CLEVELAND, OHIO, 1905-1910.

Population, Garbage Tonnage and Capital Outlays.

	1905.	1906.	1907.	1908.	1909.	1910.
Population	468,000	486,000	506,000	525,000	543,000	560,663
Garbage collected, total tons	30,382	34,891	37,606	41,247	44,590	44,747
Tons per capita per year	0.065	0.072	0.074	0.079	0.082	0.080
Total cost of works to end of current year	$70,495	$85,328	$190,830	$223,782	$225,551	$232,108

Yearly Expense and Income.

	1905. Total.	1905. Per ton of garbage.	1906. Total.	1906. Per ton of garbage.	1907. Total.	1907. Per ton of garbage.	1908. Total.	1908. Per ton of garbage.	1909. Total.	1909. Per ton of garbage.	1910. Total.	1910. Per ton of garbage.
Cost of Operation of Reduction Works—												
Labor	$26,735	$0.880	$43,732	$1.253	$44,185	$1.175	$45,625	$1.106	$40,417	$0.906	$43,526	$0.973
Coal	10,974	0.361	19,980	0.573	16,370	0.435	3,692	0.090	949	0.021	4,585	0.102
Natural gas	....	....	....	....	....	....	19,790	0.480	24,640	0.552	20,682	0.462
Superintendent and clerk	3,895	0.128	3,363	0.096	3,497	0.093	3,950	0.096	4,000	0.090	3,786	0.085
Repairs and renewals	7,109	0.234	4,764	0.136	8,076	0.214	7,667	0.186	11,242	0.252	7,307	0.163
Press cloth	1,290	0.042	2,565	0.074	3,161	0.084	813	0.020	....	....	....	....
Press racks	231	0.008	806	0.023	966	0.026	357	0.008	....	....	....	....
Insurance	314	0.011	288	0.008	187	0.005	353	0.008	....	....	....	....
Office supplies	125	0.004	146	0.004	386	0.010	398	0.009	396	0.009	199	0.004
Miscellaneous expenses	1,921	0.063	5,113	0.147	5,332	0.142	4,880	0.118	3,559	0.080	2,184	0.049
Taxes	....	....	346	0.010	459	0.012	642	0.016	714	0.016	....	....
Commission, analysis, etc.	....	....	826	0.024	472	0.013	285	0.007	343	0.008	524	0.012
Freight on product	1,855	0.061	1,455	0.042	2,203	0.059	640	0.016	....	....	124	0.003
Gasoline	....	....	....	....	....	....	....	....	2,486	0.056	2,390	0.053
Unclassified	3,157	0.104	75	0.002	171	0.005	713	0.017	97	0.002	187	0.005
Total	$57,607	$1.896	$83,459	$2.392	$85,465	$2.273	$89,804	$2.177	$88,843	$1.992	$85,494	$1.911
Interest and Depreciation—												
Interest at 4% on cost of works	2,820	0.093	3,413	0.098	7,633	0.203	8,951	0.217	9,022	0.202	9,284	0.207
Depreciation charged off	3,154	0.104	3,918	0.112	9,902	0.263	18,093	0.439	16,156	0.363	16,591	0.371
Total Cost of Operation, Including Interest and Depreciation, But Exclusive of Freight on Raw Garbage	$63,581	$2.093	$90,790	$2.602	$103,000	$2.739	$116,848	$2.833	$114,021	$2.557	$111,369	$2.489
Freight on raw garbage	4,561	0.150	5,285	0.151	5,824	0.155	6,225	0.151	6,587	0.148	6,260	0.140
Total Cost of Operation, Including Interest, Depreciation and Freight on Raw Garbage	$68,142	$2.243	$96,075	$2.753	$108,824	$2.894	$123,073	$2.984	$120,608	$2.705	$117,629	$2.629
Gross Income From Product—												
From sale of product	$57,336	$1.888	$96,351	$2.761	$116,324	$3.093	$143,711	$3.484	$148,855	$3.338	$173,[illegible]	$3.874
From inventory of product	5,563	0.183	8,694	0.250	11,979	0.319	2,582	0.063	11,242	0.252	16,[illegible]	0.376
Total Gross Income From Product	$62,899	$2.071	$105,045	$3.011	$128,303	$3.412	$146,293	$3.547	$160,097	$3.590	$190,[illegible]	$4.250
Net Income—												
Total net income before deducting freight on raw garbage	$682*	$0.022*	$14,255	$0.409	$25,303	$0.673	$29,445	$0.714	$46,076	$1.033	$78,[illegible]	$1.761
Total net income after deducting freight on raw garbage	$5,243*	$0.173*	$8,970	$0.258	$19,479	$0.518	$23,220	$0.563	$39,489	$0.885	$72,[illegible]	$1.621

*Deficit.

Results at Columbus.

The municipal garbage-reduction works at Columbus were designed and built by the city and have a capacity of handling a total of 160 tons of garbage per day. The works were completed in 1910, as previously mentioned, and have been in operation somewhat over a year. As originally built, equipment was not provided for extracting grease by means of the percolating process, but the city is now installing such equipment and the same will be ready for use in the near future.

The reduction works are located a few miles south of the center of the city and the garbage is transported in cars to the works from a loading station centrally located in the city. The total cost of construction of the reduction works proper, exclusive of land and engineering, was $189,542. Adding to this the cost of land, loading station, garbage cars, railroad siding and engineering, namely, $46,028, gives a total expenditure of $235,560. The amount of garbage handled at these works for the first six months of 1911 was 7,066 tons, corresponding to a production of 0.075-ton per capita per year. This figure is somewhat lower than the actual production per capita per year, as the production for the year, which would include the summer months, would be more than twice that for the first six months.

The cost of operation of, and net income from, the reduction works at Columbus for the six months, January to June, 1911, inclusive, is given in Table IV. In this table is included interest on the investment at the rate of 4% per annum and also a sinking fund charge of 1.78% per annum to retire the bonds in 30 years, with interest on sinking-fund payments at the rate of 4% per annum.

TABLE IV.—COST OF OPERATION OF AND INCOME FROM THE GARBAGE-REDUCTION WORKS AT COLUMBUS, OHIO, FOR THE SIX MONTHS, JAN.-JUNE, 1911.

Population, Garbage Tonnage and Capital Outlays.	
Population	188,000
Garbage collected—	
Total tons	7,066
Tons per capita per year	0.075
Total cost of works	$235,560

Yearly Expenses and Income.	Total.	Per ton of garbage.
Operation of Reduction Works—		
Supervision and payrolls	$9,724	$1.376
Coal	2,026	0.287
Electric power	844	0.120
Repairs and renewals	849	0.120
Supplies	348	0.049
Office expense	185	0.026
Miscellaneous	192	0.027
Unpaid bills	839	0.119
Total	$15,007	$2.124
Interest and Sinking Fund—		
Interest at 4% on cost of work.	4,711	0.667
Sinking fund at 1.78% on cost of work	2,097	0.297
Total cost of operation, including interest and sinking fund, but exclusive of freight on raw garbage	$21,815	$3.088
Freight on raw garbage	2,325	0.329
Total cost of operation, including interest, sinking fund and freight on raw garbage	$24,140	$3.417
Gross Income From Products—		
From sale and inventory of product	$29,616	$4.192
Total gross income from product.	$29,616	$4.192
Net Income—		
Total net income before deducting freight on raw garbage.	$7,801	$1.104
Total net income after deducting freight on raw garbage.	$5,476	$0.775

From an examination of this table, it will be seen that the net income, after deducting the freight charges on the raw garbage, for the first six months of 1911, was $5,476, or at the rate of $10,952 per year. While these works do not show at the present time as great a net annual income proportionately as the Cleveland works, it should be stated that the Columbus works are operating at very much less than their rated capacity and that the fixed charges are correspondingly high. Further, a much greater weight of garbage can be handled with but a slight increase in the cost of labor. It is, therefore, only reasonable to infer that the net profits with an increased amount of garbage handled will be considerably increased.

The grease recovered during the first six months of 1911, without the use of the percolating process, was 2.57% and the tankage 15% of the weight of the garbage reduced. Careful analyses were made of the dry tankage shipped from the works during this period, which analyses showed that the dry tankage still contained approximately 10% of grease. From this tankage, approximately 8% of grease is recoverable, and it is to recover this grease that the percolating process is being installed. Had the percolating process been in use during the six months period cited, the grease recovered would have been about 3.8% and the dry tankage about 15% of the weight of garbage reduced.

The average price received for grease at Columbus during the six months was 5.19 cts. per lb. and for dry tankage $10.28 per ton. The higher price received for the tankage is due to the fact that the liquids, from which the grease has been separated, are evaporated down to a syrup and this syrup mixed with the dry tankage, after which the tankage is redried, the result being a tankage of a much better grade than that ordinarily produced in reduction works.

The writer has attempted to present the figures herein given in an entirely impartial manner, and their presentation is not to be construed as an argument either for or against reduction. Under certain conditions it may be found that the disposal of garbage at reduction works will offer the best solution, whereas under other conditions it may be found that incineration will offer the best solution. Further, under still other conditions, a combination of the two may be found more desirable.

Some Observations Upon the Collection and Disposal of Garbage.

Under the caption given just above, Mr. Samuel A. Greeley, Jun. M. Am. Soc. C. E., sent a paper to the annual meeting of the New Jersey Sanitary Association at Lakewood, on Nov. 24. Owing to the fact that Mr. Greeley had recently gone to South America, the paper was read by Mr. Rudolph Hering, M. Am. Soc. C. E.

The paper was not at all technical in character, as was fitting for the occasion. Since, however, it embodies in part observations made by Mr. Greeley during two trips to Europe, and also during his experience, first as Engineer-in-Charge of the refuse incinerator of Milwaukee, and later as Superintendent of garbage collection and disposal, we give a brief outline of his paper.

In opening, the author called attention to the fact that at Milwaukee there have been only two or three complaints regarding disposal by incineration since the furnaces were put in operation, as contrasted with an average of 20 or 30 complaints a day regarding collection. This contrast was drawn to emphasize the fact that in some respects refuse collection to-day is more troublesome in its routine operation than is disposal.

Taking up in some detail the question of collection and beginning at the house and alley, Mr. Greeley remarked that from many points of view the one-can system is preferable to the system which calls for a separation of garbage and other refuse and placing it in two or three cans. When all the refuse is mixed the odors from garbage are lessened, the dust from ashes is decreased and at the same time the weight of garbage and ashes combined holds down papers and other light refuse which cause a nuisance by blowing about. The system of mixing refuse also tends to lessen the opportunities for the development of flies. The one-can system is favorable to the maintenance in cleanly condition of the wagons used for collecting garbage.

Altogether, the one-can, or no-separation, system gives most satisfactory results at the house and, as it results in no great increase in the cost of final disposal, little objection can arise on the score of cost.

Taking up next problems of collection, the author stated that where the garbage is collected separately, daily collections may be necessary in summer, but if the garbage is mixed with other refuse collections made three times a week are necessary.

Refuse wagons should be as large as possible, especially where the haul is long. The wagons should be washed daily in warm weather. In Germany they are sometimes washed after each trip.

In the United States, probably more garbage is disposed of on land than by any other method, but notwithstanding this fact one rarely finds evidence of money having been appropriated for the care of garbage dumps.

Feeding garbage to pigs is common in New England. Sometimes this is the only food the pigs receive, while in other cases it is merely the basis for food, the garbage being disinfected by steaming. In general, feeding to pigs is not a cleanly method of disposal, but it yields a small profit. Where the garbage is fresh, this system of disposal works well.

Progress in garbage reduction has been made of late. Formerly, reduction was less satisfactory than now, because the work was always done under private contract for short periods.

As to incineration, Milwaukee was cited as an example of a city where the two-can system of collection prevails. The garbage of the whole city is burned, but only sufficient refuse to burn the garbage is hauled to the incinerator and that, of course, is taken from the districts nearest the furnaces. A 600-KW. generator is now being installed at the Milwaukee incinerator to utilize the steam raised by means of the heat of incineration. The electric power will be used to pump water for river flushing. It is estimated that this will effect an equivalent of a saving of $24,000 a year. A crushing outfit is also being installed to aid in the utilization of the clinker from the incinerator.

In general, every local garbage disposal problem should be given careful study in order to meet local conditions. In large cities a combination of reduction and incineration may be advisable.

The only discussion on this paper was by Mr. Morris R. Sherrerd, M. Am. Soc. C. E., of Newark, who stated that probably in many cities a combination of incineration and reduction would be better than one of these processes alone. The proper separation of garbage from refuse by householders is an obstacle to the success of reduction plants. A further difficulty lies in the inefficient class of help which alone is available to the contractor engaged in garbage collection. This sort of help cannot be relied upon to carry out instructions, and whenever any disorder in the collection system arises the collectors mix the different classes of refuse. This being seen by the householders, they become careless of their separation of the refuse into classes. If good separation can be secured, Mr. Sherrerd said in conclusion, reduction would seem to be cheaper than incineration with heat utilization.

The Japanese Torpedo-Boat Destroyer, "Harusame," sank, Nov. 24, off Shima Province, Japan. Forty-five men of the crew of 60 were drowned.

A Boiler Explosion at Liverpool, England, Nov. 24, killed 33 persons and injured about 100 others. The explosion occurred at the oil-cake mills of J. Bibby & Sons. One of the mill buildings was wrecked.

A Railway Bridge Collapsed in France, Nov. 23, at Montreuil-Bellay, carrying down with it a passenger train into the Thouet River. Reports state that about 30 of the 100 passengers were drowned. The accident is ascribed to the undermining of one of the bridge piers by a flood.

The Water and City Electric Lighting Services of Seattle, Wash., were cut off on Sunday, Nov. 19, 1911, by "unprecedented floods" in the Cedar River. Large portions of the city were entirely deprived of their regular water service, while others were left dependent on a relatively scant supply stored in reservoirs, but which was likely to fail before the Cedar River supply could be restored. It became necessary to close schools, shut down elevators and cut off or curtail the water-supply of many industrial plants. The

electric lighting service, it appears, was quickly made good by current from a private plant. The cause of the floods was a combination of melting snows and heavy rains for two days. The resulting flood poured over the storage and power dam at Cedar Lake (some 40 miles above the city) at a reported height of 8 ft. 7 ins. above the crest of what appears to have been wooden flashboards built on the masonry dam some three years ago. This superstructure was carried away, the two pipe lines supplying the power-house were broken, and, still further down stream, a special bridge at Landsberg, supporting the two water-supply conduits, was washed out, taking with it some 180 ft. of the conduits, and thus entirely cutting off the city's water-supply. These conduits were of continuous wood-stave pipe, one 60 and one 42 ins. in diameter—only the larger one being in service at the time. Work was speedily begun on a temporary bridge and the closure of the gap in the conduits; also on the installation of temporary pumps to take water from Lake Washington, within the city, and from Swan Lake, some distance up the supply conduits. City officials are quoted in the Seattle press as affirming and railway officials as denying that the pipe-line-bridge washout was due to a rectification of the river course above the bridge, by the new Milwaukee line which was built into Seattle a few years ago. It is said that this change in the river carried the flow directly against the two steel cylinder piers which supported one line of the bridge. It is also stated that this same bridge was thought to be in danger during a flood two years; also that when the conduits were laid out consideration was given to driving a 4,000-ft. tunnel to avoid the river crossings—of which there is a second, besides the one washed out. A half score of the leading city officials of Seattle worked night and day to restore the water and lighting services—particularly the water service. These included Mr. A. H. Dimock, City Engineer, and his assistant, D. W. McMorris; L. B. Youngs, Superintendent of Water-Works, and J. D. Ross, Superintendent City Lighting Department. Dr. J. E. Crichton, Commissioner of Health, and Mr. Geo. W. Dilling, Mayor of the city, were each active in their respective capacities. Mr. R. H. Thompson, for many years city engineer of Seattle, also came to the aid of the city.

A New Newark-New-York City Electric Railway was opened on Nov. 26, 1911, by the Pennsylvania R. R. The road extends from a new terminal station in the city of Newark, across the Passaic River, the Hackensack Meadows and the Hackensack River on a new line and joins in Jersey City the subway line of the Hudson & Manhattan Ry., on whose tracks it crosses under the Hudson River into the terminal station at Fulton St., New York, and, by the uptown line of the same company, under Sixth Ave. to 33rd St. Multiple unit trains operating on third-rail transmission, will be run on about a ten-minute schedule between the two cities, at the regular fares now prevailing for the steam railways.

A Subway in Toronto, Ont., has been designed by the city authorities and bids for its construction were received on Nov. 1. The purpose of these bids was to enable the city to arrive at the actual cost of a subway system so that a by-law to provide the necessary money might be prepared for submission to the taxpayers for their approval. On such approval the contract will be awarded.

The subway is about three miles long and extends north from the lake through the business portion of the city to the city limits. The designs call for a reinforced-concrete structure, carrying two tracks. The bids were taken on six separate sections and the totals for the six sections by any one bidder ranged between $2,475,000 and $5,195,000, with an average about $3,500,000.

A Ballast Sprinkling Car was described in a committee report presented at the annual meeting of the Roadmasters and Maintenance of Way Association of America. The description is substantially as follows:

Great discomfort and inconvenience to passengers on fast trains has been experienced where stone ballasting operations are in progress, owing to the dust in the stone, particularly in continued dry weather when meals were being served in dining cars. A new and very inexpensive remedy has been found, in mounting two tanks of 4,000-gals. capacity on the standard Rodgers ballast spreader car and sprinkling the stone with water while it is being unloaded. The tanks are placed on each end of the car and are provided with a sprinkler arrangement similar to a street sprinkler. A pipe connection between the tanks permits the water to lower in both tanks at the same time, and only one tank need be placed at the water column when filling them. The water is started as soon as the plow commences to plow the stone out. It entirely eliminates the dust in the trains and washes and cleans the stone so that it makes it better to handle and is quite inexpensive compared with the benefit derived.

Personals.

Mr. Robert E. Kremers, formerly Engineer of Buildings of the Oregon Short Line R. R., has been appointed Engineer of Construction of the Building Department of the City of Portland, Ore.

Mr. Ernest R. Bissell, Division Superintendent of the Lake Erie & Western R. R. at Lafayette, Ind., has been appointed Division Superintendent of the Lake Shore & Michigan Southern Ry. at Toledo, Ohio.

Mr. L. E. Perry, Jun. Am. Soc. C. E., has resigned as Secretary and Engineer of the Houston Perry & Co., Seaford, Del., to accept a similar position with the Perry-Bradley Lumber Co., of Salisbury, Md.

Mr. Alfred H. Terry, Assoc. M. Am. Soc. C. E., formerly of the real estate department of the New York, New Haven & Hartford R. R., at New Haven, Conn., has been made City Engineer of Bridgeport, Conn.

Mr. Julius Kruttschnitt, Vice-President and Director of Maintenance and Operation of the Harriman lines, has moved his residence from Chicago, Ill., to New York City, which will hereafter be his headquarters.

Mr. Arthur H. Dimock, M. Am. Soc. C. E., Assistant City Engineer of Seattle, Wash., has been appointed Acting City Engineer to succeed Mr. R. H. Thomson, M. Am. Soc. C. E., who will devote his whole time to his office as Port Engineer of the Port of Seattle.

Mr. J. R. Cameron, formerly General Superintendent of the Canadian Northern Ry., has been promoted to Assistant General Manager of the same. Mr. A. Wilcox, formerly Superintendent at Port Arthur, Ont., has been appointed General Superintendent of the Western division, with headquarters at Edmonston, Alberta; and Mr. A. E. Warren, formerly Superintendent at Winnipeg, Man., has been appointed General Superintendent of the Central division at Winnipeg.

Mr. A. W. Martin, Superintendent of the Boston division of the New York, New Haven & Hartford R. R., has retired after 44 years of railway service. He began his railway career in 1867, when 16 years old, as a freight clerk on one of the early New England railways, which later became a part of the New Haven system. He rose through various clerical positions to his present position in 1908. He is succeeded by Mr. F. S. Hobbs, formerly Freight Trainmaster at Boston, Mass.

Mr. A. M. Sutherland, for 22 years Assistant Superintendent of the Duluth, South Shore & Atlantic Ry., has retired from railroading after 50 years of active work. He entered the railway service at the age of 17 as a track laborer on the Grand Trunk Ry. Six years later he became a section foreman on the Chicago & Northwestern Ry. Later he was successively locomotive fireman, locomotive engineer, and roadmaster of the same railway. In 1889 he was made Assistant Superintendent of the Duluth, South Shore & Atlantic Ry.

Obituary.

M. H. Elkin, Division Engineer of the Lehigh Valley R. R., died at his home in Hazleton, Pa., on Nov. 26.

E. D. Nash, for a number of years Assistant Superintendent of the Southern division of the Central Vermont R. R., at New London, Conn. died in Panama on Nov. 18, following an operation for stomach trouble.

John J. Ryan, Superintendent of Motive Power of the Louisiana Western, the Galveston, Houston & Northern, the Galveston, Harrisburg & San Antonio and the Texas & New Orleans Rys., died on Nov. 21 at his home in Houston, Tex. He was born July 1, 1849, at Boston, Mass., and h's early education was received in the public schools of that city. While still a lad he worked as an apprentice at the McKay & Aldus Locomotive & Marine Works. His railway service began in 1879 with the New York & Lake Erie R. R. After numerous changes he became in 1877 a machinist with the Galveston, Harrisburg & San Antonio Ry., from which position he rose in six years to be Master Mechanic. Ten years later he was Superintendent of Motive Power of this and the other three railways mentioned above, which comprise the Atlantic System of the Southern Pacific Ry.

Engineering Societies.

COMING MEETINGS.

AMERICAN SOCIETY OF REFRIGERATING ENGINEERS.
Dec. 4. Annual meeting at New York City. Secy., William H. Ross, 154 Nassau St., New York City.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Dec. 4-9. Annual meeting at Havana, Cuba. Secy., Wm. C. Woodward, Washington, D. C.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
Dec. 5-8. Annual convention at New York City. Secy., Calvin W. Rice, 29 West 39th St., New York City.

NATIONAL IRRIGATION CONGRESS.
Dec. 5-9. Annual session at Chicago, Ill. Secy., Arthur Hooker, 214 Home Life Bldg., La Salle, Chicago, Ill.

ASSOCIATION OF AMERICAN PORTLAND CEMENT MANUFACTURERS.
Dec. 11-13. Annual meeting at New York City. Secy., Percy H. Wilson, Land Title Bldg., Philadelphia, Pa.

AMERICAN INSTITUTE OF ARCHITECTS.
Dec. 12-14. Annual convention at Washington, D. C. Secy., Glenn Brown, the Octagon, Washington, D. C.

AMERICAN INSTITUTE OF CHEMICAL ENGINEERS.
Dec. 20-22. Annual meeting at Washington, D. C. Secy., J. C. Olsen, Polytechnic Institute, Brooklyn, N. Y.

AMERICAN SOCIETY OF AGRICULTURAL ENGINEERS.
Dec. 27-29. Annual meeting at St. Paul, Minn. Secy., J. B. Davidson, Ames, Iowa.

AMERICAN ASSOCIATION FOR THE ADVANCEMENT OF SCIENCE.
Dec. 27-Jan. 3. Annual meeting at Washington, D. C. Secy., L. O. Howard, Smithsonian Institution, Washington, D. C.

PACIFIC NORTHWEST SOCIETY OF ENGINEERS.
Jan. 6. Annual meeting at Seattle, Wash. Secy., Joseph Jacobs, 803 Central Bldg., Seattle, Wash.

AMERICAN SOCIETY OF ENGINEERING CONTRACTORS.
Jan. 9. Annual meeting at New York City. Secy., J. R. Wemlinger, 13 Park Row, New York City.

MICHIGAN ENGINEERING SOCIETY.
Jan. 9-11. Annual meeting at Lansing, Mich. Secy., Alba L. Holmes, 574 Wealthy Ave., Grand Rapids, Mich.

ENGINEERS' SOCIETY OF WESTERN PENNSYLVANIA.
Jan. 16. Annual meeting at Pittsburgh, Pa. Secy., Elmer K. Hiles, 2511 Oliver Bldg., Pittsburgh, Pa.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 17-18. Annual meeting at New York City. Secy., Charles Warren Hunt, 220 West 57th St., New York City.

SOCIETY OF AUTOMOBILE ENGINEERS.
Jan. 18-20. Annual meeting at New York City. Secy., Coker F. Clarkson, 1451 Broadway, New York City.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Jan. 23-25. Annual meeting at New York City. Secy., W. W. Macon, 29 West 39th St., New York City.

OHIO ENGINEERING SOCIETY.
Jan. 24-26. Annual meeting at Cleveland, Ohio. Secy., Clyde J. Knisely, New Philadelphia, Ohio.

Engineering Association of the South.

At the annual convention in Atlanta, Ga., Nov. 17-18, officers for the coming year were elected as follows: President, Arthur Pew, Atlanta, Ga.; Secretary and Treasurer, J. C. Evans, Nashville, Tenn.; Member of Board of Directors, Hunter McDonald, Nashville, Tenn. The membership of the association is now 349, representing 17 different states and four foreign countries. There were 82 members in attendance at the convention.

New Jersey Sanitary Association.

The 36th Annual Meeting of this Association was held at Lakewood, N. J., Nov. 24-25. Of the papers presented, particular mention may be made of one by Mr. Morris R. Sherrerd, M. Am. Soc. C. E., Consulting Engineer to the New Jersey State Water Supply Commission, this paper outlining "Some Features of the Larger Water-Supplies of the State." The other paper was entitled "Some Observations Upon the Collection and Disposal of Garbage." It was prepared by Mr. Samuel A. Greeley, Jun. M. Am. Soc. C. E., recently of Milwaukee and now temporarily engaged in engineering work in South America. A brief abstract of Mr. Greeley's paper appears in this issue.

Prof. John B. Smith, of New Brunswick, was elected President of the Association for the ensuing year. Dr. James A. Exton, of Arlington, was re-elected Secretary and Mr. Geo. P. Olcott, of East Orange, was re-elected Treasurer. Mr. Clyde Potts, M. Am. Soc. C. E. (New York City address, 30 Church St.) was elected Chairman of the Executive Council.